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## The Public Roads of the United States.

About ten years ago there was considerable controversy concerning the approximate length, cost of construction and maintenance charges of public roads in this country. Some figures were given out by government officials regarding the advantages to farmers of public roads of fairly good construction, and these figures were attacked as extremely inaccurate, and also for the illogical use made of them. The tilt between Prof. I. O. Baker of the University of Illinois and the late Roy Stone will be recalled with considerable interest by those who followed this controversy. Probably the figures were far from accurate, but they had a certain measure of value as attempts in the direction of securing information on which to base some decidedly important studies. It goes without saying that the federal government is best equipped to gather information of this nature, and Mr. L. W. Page, director of the Office of Public Roads of the Department of Agriculture recognized this fact some time ago and arranged for another and better investigation, covering the mileage, revenues and expenses of public roads in 1904 throughout the country. By his enthusiasm in public road affairs he was able to interest officials and private individuals to help him in this work, and the result, just made public, is an interesting compilation of data far more reliable than any previously gathered in this country.

The collection of the statistics has been a troublesome task, not only on account of the great diversity which exists in the methods of carrying on road improvements in different parts of the country, but also because it was necessary in some cases to write from fifteen to twenty letters to a single township in order to secure all the information needed to make the investigation complete. In some states there are as many as 1,500 of these townships which must be taken into account. In states where road taxes are assessed and collected by county officials, the task was comparatively easy, but that it was an onerous undertaking is shown by the fact that about 60,000 communications had to be sent out before the investigation was complete. The systems of road taxation and the methods of collecting and sending road funds differed so radically in various states and even in parts of the same state that it was necessary to prepare a great variety of blank forms before all the data could be obtained. After they were secured the accuracy of all doubtful figures was verified so far as practicable. Some reports were returned to those making them for checking or correction, and in some states the heads of the highway and geological departments rendered valuable aid in revising the figures. It is possible, however, that errors crept in, for the mileage of roads on the boundary lines of townships and counties may have been reported twice in some cases and not at all in others. The roads in many counties and townships have never been surveyed and estimates had to be made of their length in such cases. In some places no permanent records have apparently been kept of the collection and expenditure of road funds and in other places the records are in a confused condition. In a few instances the officials refused to supply the information unless paid by the Agriculture Department, and as the Department had no funds for the purpose, it became necessary to secure the information through unofficial sources. Taking the figures as a whole, however, it is believed that they are fairly correct and will form a valuable basis for comparison and future work.

The report uses the term "improved road" as signifying a highway graded, drained and surfaced with a material or combination of materials or treated with the application of some preparation resulting in a reasonably smooth, firm and durable surface. The figures show that in 1904 there were 2,161,570 miles of public road in the United States of which amount 7.14 per cent. had been improved as above defined. Of these improved roads 108,233 miles were surfaced with gravel, 38,622 miles with stone, and 6,810 miles with special materials like shells, sand-clay, oil and brick. Taking the country as a whole, there is 0.73 miles of road per square mile of territory, or 1 mile of road to every 35 inhabitants and 1 mile of improved road to every 492 inhabitants. These figures do not include roads in the Indian Territory, Alaska, and the island possessions or the mileage of roads in incorporated cities and villages.

The District of Columbia stands first with 3.18 miles of road per square mile of area, and Connecticut is next with 2.9 miles. Rhode Island has 2.24 miles, Pennsylvania 2.21 miles and Arizona has but 0.05 mile. The District of Columbia has 1,459 people per mile of road, Rhode Island 181 people, Massachusetts 164, New Jersey 127 and Connecticut 64. Nevada has but 3 persons per mile of public road, North Dakota 5, South Dakota 7, Wyoming 8, Idaho and Oklahoma 9 each and Montana 10. These figures relate to public roads of every sort. When the improved roads alone are considered, Indiana is found to have the largest mileage, 23,877 miles. Ohio has 23,460 miles, Wisconsin 10,633 miles, Kentucky 9,486, California 8,803, Illinois, Massachusetts and Michigan over 7,000 miles each, Minnesota over 6,000 miles, and New York over 5,000 miles. In about two-thirds of the states gravel has been the principal surfacing material used in improving the roads. Massachusetts has 45.89 per cent. of the roads improved in some manner, Rhode Island has 43.26 per cent., Indiana 34.94 per cent., Ohio 33.78 per cent., California 18.87 per cent., Connecticut 16.75 per cent., Wisconsin 16.72 per cent., Kentucky 16.60 per cent., New Jersey 16.32 per cent. It is interesting to observe how closely the higher percentages of improved roads agrees with the higher figures of population per mile of road. Unquestionably this fact is the kernel of the growing regard with which railroad corporations are holding highway improvements. Improved highways are becoming the best feeders for railways, and the heads of great railway systems are now seriously directing their efforts toward securing the betterment of common roads.

The feature of the investigation which is most important, however, is not that relating to the extent of the improvements, but rather to the amount of the expenditures for highway work in different parts of the country and the returns obtained from these sums. The investigation shows that during 1904 the public roads were responsible for an expenditure from property and poll taxes, bond issues and state-aid funds, together with the value of labor expended under the statute-labor laws, of nearly $80,000,000. Of this amount nearly $54,000,000 was obtained from property and poll taxes payable in cash, nearly $20,000,000 was the value of the labor taxes, and the remainder came from bond issues and state-aid funds. The expenditure amounted to $37.07 per mile of public road, or $1.05 per capita. These figures include the cost of such bridges as were necessary in connection with road improvements. Although the sum is small when it is divided on the basis of the total mileage of public highways, it is about eight-thirteenths of the expenditures for public schools in the country during 1903-1904, and about ten-thirteenths as much as was spent for river and harbor improvements.

The voluminous statistics which the Office of Public Roads has just made public shows conclusively that the best results will be obtained in those states where all road taxes are paid in cash, in order that the work may be placed in the hands of persons who have a practical knowledge of the subject and whose duty it is to devote their whole time and attention to highway improvement. New York and Pennsylvania have encouraged this practice by giving over to all counties where the practice of working out road taxes has been abolished in favor of cash payment, a certain percentage of the amount of the taxes so collected. Taking the country as a whole, the figures show that if the percentage of improved roads is a fair standard for judging the condition of a district, the best results have been obtained in those states which have abolished statute labor and where the road taxes are paid chiefly in cash. The amount of property taxation for road purposes varies widely, and one of the best features of the investigation referred to is the compilation of information concerning what is done in this respect throughout the country. If the investigation produced no other result than this comprehensive statement of taxation for road purposes, it would have been fully justified. It is impracticable to summarize it here and the reader is referred to the full report for it. The document is an important contribution to our knowledge of roads, and the Office of Public Roads is to be congratulated on the success of its investigation, from which a good many people did not expect any such useful and instructive results.

## Operating Records in New Power Plants.

In the construction of a new power plant for important service the installation of the machinery usually takes so much attention from the operating staff that there is little opportunity for the keeping of records in great detail. Everyone realizes that it is useless to expect good economy in fuel consumption and labor cost per kilowatt-hour before the plant is half loaded, and for this reason the question of log sheets and load curves is generally one of the last things to be put on a regular working basis. In some plants practically no records are kept until the station is opened for commercial service.

The wisdom of not attempting too great refinement in early records is indisputable, but it is clearly a mistake to go to the other extreme and keep practically no station log whatever. For example, in the case of a power plant approaching completion in connection with the electrification of a steam railroad, steam is sure to be demanded from the boilers long before commercial trains begin to run; engines or turbines must be turned over and gradually speeded up to full usefulness; and some of the generating units will be finished in installation before others and will be needed for the operation of trial trains on experimental track sections, or for supplying current for construction purposes. In an industrial plant designed to supply power to a motor-driven factory installation certain departments will be first equipped, tested and commercially supplied, current being required more or less irregularly each day or two until the changeover is to a considerable extent completed. If no records of fuel consumption by days, general load variations and partial or complete shifts of attendance are kept, the company may know the grand total cost of getting the plant ready for service, but it will not know anything about the behavior of the installation on the cost side in relation to the exigencies of irregular though practically continuous service.

The main points of operation should be noted day by day during the construction period quite as much as in the routine days to follow. It ought to be possible to keep a close record of the coal burned each day, the total station output each day, the water consumption and the hours of attendants put in on operating work as distinguished from inspection of contractors' doings. It is a nice question in electrical auditing to determine just what part of a plant's running expenses during the installation period should be charged to construction, and what part to the operation of the service, as in the case of supplying trial trains with electric power. Separate meters are valuable in this connection, even if their use is temporary.

It is often claimed by the owners of new plants that the load curves imposed upon the station by the inauguration of a partial service are not representative, and therefore are of little interest. Few plants can show a daily load curve from the first hour in which they supplied current for actual use down to the present, but it is a fact that the study of some of these freakish diagrams in cases where they have been taken has proved to be of the greatest value from an engineering standpoint. It is distinctly the conception of the layman that matters must be well established in an installation before the results of operation will be instructive. Of course, deductions from incomplete data are a positive hindrance to good work, but if all the essential conditions are known, their analysis is bound to be suggestive, no matter how wide may be the departure of the data from the ordered regularity attained in routine service. The chief engineer may not have time to plot load curves himself during the construction period, but if the essential readings are taken and recorded, the deductions can be made at leisure. It is better to find out that a given machine needs modifications to secure economy before commercial service begins than afterwards. Just how much attention should be given by the operating staff of a station to construction and installation pure and simple is a question; practice in the past has neglected operating problems in the early stages in favor of assembly and erection oversight, but in the future it is probable that more consideration will be paid to the broader questions of plant economy. If the company's organization permits, its regular engineering staff rather than the operating men in the station itself should be charged with the duty of seeing that specifications and guarantees are met by contractors.

No general rules can be laid down as to the form of records best adapted to the needs of a plant in the early stages of its work, but a little foresight in each case will enable the essentials to be properly recorded. If a standard size of daily, weekly or monthly log can be determined at the ouset much will be gained even though the forms of charts and tabulations may necessarily be varied and extended later. The great point is to have the records filed consistently with one another, leaving ample space for future contingencies. The deductions may be irregular or postponed, but if the facts are at hand, the company will never regret securing them during the days when everything in the plant from the coal-handling machinery to the high-tension lightning arresters was in a state of upheaval. Fuel consumption, labor, water and supplies will show extravagant figures per unit of output with the abominable load factors which irregular outputs in the construction period throw upon the plant, but they point the way toward a more efficient service as the duty of the station broadens out into routine service. It would be an instructive thing to plot the curve of coal consumption per kilowatt-hour from the early weeks of a new plant's service down to the months of regular operation on commercial loads, taking the fuel as the ordinate and time as the abscissa. Yet, as suggested above, few plants have the data upon which such a curve could be built. Such will not always be the case, for the time is coming when engineers will study the abnormal conditions of early service with the same interest that they give to the records of plants which have been turned over to the owners for the rest of their commercial lives. A special reason for keeping such records is their probable usefulness when something goes wrong during operation. It is well known that those who become acquainted with the behavior of a plant while it is being tuned up are generally able to locate promptly the cause of any troubles that subsequently arise.

## The Maintenance of Small Works for Treating Sewage and Water.

The rapid increase in the number of small works for the treatment of water and sewage is noticeable, and it is therefore timely to sound a note of caution regarding the results to be expected from such plants. Practically all of them depend for their efficiency upon the properly controlled action of bacteria, and this action is affected by a large number of different conditions, some of which we are fairly well acquainted with while others present an almost untouched field of research. Most of the larger plants for purifying water and treating sewage are operated in a fairly efficient manner, and some of them receive excellent attention. This is not true, however, of the smaller plants, and as the total investment in them is probably much larger than that in the works built for great cities, the importance of a better appreciation of the responsibility for their efficient management is evident. For some reason it is comparatively easy to secure an appropriation for the construction of such works for small cities, but extremely difficult to have the plants managed properly after they are built. In fact, there are so many sewage disposal works in the Central States which are now practically useless on account of mismanagement that it is somewhat surprising more plants are being constructed all the time. It would seem as though the poor results from these mismanaged plants would stop the progress of sewage disposal. This would have happened, probably, were it not that public health authorities generally have power to compel small cities and towns to put in sewage disposal works, although unfortunately they have not been given authority to compel the owners of such plants to operate them in an efficient manner. It is strange that a town should be willing to spend a considerable sum in proportion to its resources for a sewage disposal or water purification plant, and then by its failure to put the works in charge of a competent man should practically destroy the value of such an investment. Yet this condition can be observed in so many places that it is a manifest fact which sanitary engineers must recognize. They are being held responsible in a considerable measure for the failure of a good many of these ill-managed plants to run as they should, and it therefore behooves them to take steps which will prevent such undesirable criticisms of their work.

There is a great difference in the character of the sewage and in the nature of the water supplies of different places, and for this reason hard and fast rules applicable to all communities have to be of such a general nature that they are useless except as somewhat superficial suggestions. Moreover, just what details will be most satisfactory in the management of sewage disposal and water purification works in any city may be open to some doubt. The most an engineer can do in designing such plants is to provide a system of works having enough flexibility so that by careful observation of the early stages of operation, the best procedure can be ascertained. When it is ascertained, the maintenance of the operations according to this method is comparatively easy. Where the character of a water supply fluctuates considerably from month to month and in those places where the sewage is liable to change in character for one reason or another, there is particular need of a careful study of the operation of the plants during the first few years they are in service. Accordingly there are good grounds for the belief that designing engineers will do well to urge their clients in some cases to make contracts with them to supervise the operation of sewage disposal and water purification plants for a sufficient period after their completion to insure a full determination of the problems that will arise and the methods of meeting them. This supervision is not necessarily an expensive matter; it may cost considerably less than the losses incurred through faulty management of the works. It does not necessitate the constant presence of one of the consulting engineers at the works, nor does it require a highly trained chemist or biologist to be in constant attendance. In most cases the same class of men who have mismanaged the plants in so many small places, owing to their ignorance, could have been trained to look after the works properly if they had been under the supervision of the designers of the works from the outset of the operation.

The remedy for the present unfortunate situation lies largely with the designing engineers. If they take the position in arguing with their clients that the works built from their plans form a delicate piece of apparatus, which must be managed with a much higher degree of attention than a machine tool or an engine, in order to produce the best results, it is but reasonable to expect that the supervision of such works will be entrusted to their designers during a sufficient time to insure the proper training of the attendants and the proper procedures in treating the water or sewage under the different conditions that arise during the year. So much criticism is being made concerning the inefficiency of works of this nature, that engineers making a specialty of such plants ought to protect themselves by urging their clients to place the works under their supervision for operation during a short time at least.

## A Possible Result of Electric Train Operation.

Elsewhere in this issue there is a description of some of the engineering features of the difficult mountain section of the Western Pacific Railway. It will be noticed in the article explaining it that there is an exceptional opportunity along the line to develop a large amount of water power; in fact, a power company has been formed for the purpose and it is currently understood to be working in harmony with the railway company. In the same way there are excellent opportunities for power development on the mountain line of the Chicago, Milwaukee & St. Paul extension to the Pacific Coast and also near the other northern trans-continental routes. It is generally believed that advantage will shortly be taken of these exceptional opportunities for developing water power to operate mountain sections of some of the roads electrically. The arrangements for electric traction were outlined in this journal on August 17, and it is unnecessary to review them at length again. The great advantage offered by the introduction of the electric locomotive is believed to be the increase of tonnage capacity of single track and the general reduction in cost per ton-mile resulting from handling heavy trains at higher speed with a certainty of operation not practicable with the steam locomotive. By properly proportioning the electric locomotive and the power station and line, it is practicable to operate trains at any speed up grade that is consistent with the alignment of the track. In other words, the same schedule can be maintained upgrade and downgrade, and the movement of trains on heavy-grade sections can thus be greatly facilitated.

There is another feature, however, of electric operation which will probably be found an important one, although it is opposite in a sense to that just stated. At the present time there is a marked demand for a more rapid movement of mixed freight. One of the serious criticisms against certain Western railroads is that even when high rates are paid for moving freight, the latter is handled only when the railroad gets ready and not with reasonable regard to the necessities of the shipper. In other words freight of every sort, except the most perishable classes, is held until a train load is secured satisfactory to the operating officials. Naturally enough on those roads which have been laid out with a view to handling business in this way, the attempts of the shippers to secure prompt deliveries by sending out lighter trains at more frequent intervals are not met with any marked degree of enthusiasm. There can be no question, however, that this matter of delay is responsible for a considerable part of the present general belief among business people and farmers that a reform can be made in some branches of railway operation without hurting the stockholders of the road and to the great advantage of the shipper.

Just how far the use of electricity will enable such a change in operation to be made is something of a problem, for it involves a good many other things than the use of electric locomotives. For instance, sidings have to be provided in suitable places for frequent short trains as well as for long heavy ones, and these sidings should not be at the same places for both kinds of operation. The amount of time a train spends on sidings when traffic is at all heavy is often so large as to be a material factor in the total time of transit of freight. It seems fairly possible, however, that electric operation on mountain grades for heavy trains, by supplying the means for equally economical operation of more frequent lighter trains, will lead to experiments with the latter. This is contrary to accepted methods of economical railway operation with steam locomotives, but if electric operation permits the defects of those methods to be avoided without appreciable sacrifice of economy, public opinion will eventually compel their adoption.

## Notes and Comment.

The Belmont Tunnel connecting Long Island City and Manhattan was traversed on Tuesday morning by an electric car carrying about forty officers of the road and the city and guests identified with New York traction matters. Later in the day, at a luncheon at the Hotel Belmont, a silver model of the car was presented to Mr. St. John Clark, chief engineer of the work, whose energetic direction of the tunneling operations is largely the cause of the fact that the first regular passenger car entering Manhattan by way of a subaqueous tunnel ran over this route. The work has been fully described in The Engineering Record of March 3, 1906, and June 8, 1907. It consists for the most part of two parallel single-track tunnels 28 ft. apart on centers and is about 1,700 ft. long. Three sections have been used, according to the material traversed. In rock the concrete lining has a clear width of 12½ ft. and a clear height of 13 ft., and is of horseshoe shape. In earth, the tubes have a segmental iron lining with a clear diameter of 15½ ft., and in Long Island about 1,800 ft. of double-track rectangular reinforced concrete tunnel has been built. Some of the shield work has been very difficult and dangerous. Where the sand was very soft, the shields were pushed ahead without excavating at all until the sand became too dense for any further movement and had to be removed in the usual way. Where it was soft, it was the custom to remove the slope under the hood of the shield as rapidly as possible and support the face by a bulkhead of horizontal boards braced against the shield, the diaphragm being backed with hay where the sand was fine enough to penetrate the cracks. Mr. Robert Shailer, chief engineer for the Degnon Contracting Co., which built the tunnel, was in personal charge of this work, and the manner in which he conducted it has been warmly commended by specialists in such construction. Mention should also be made of the expedient to prevent distortion of the iron lining by temporary unbalanced pressures. Each alternate ring, as soon as set, was provided with a 1¼-in. horizontal tie with sleevenut adjustment; this was pinned to the flanges of the ring at the springing line, and screwed to a moderate initial tension, holding the lining in shape until the earth settled firmly about it or grout could be injected behind it. Less has been said about this work than any other of the subaqueous tunnels entering the city, and most of the attention it has received has been due to controversy concerning the franchise under which operations have been conducted. When the terminal facilities are completed it will open up a rapid transit route into the heart of Manhattan which should play an important part in the development of the Borough of Queens, a section of the city which has been developing slowly on account of the absence of adequate communication with Manhattan. The location of this borough is such that the completion of the Belmont tunnel, the Blackwell's Island bridge and the Pennsylvania tunnels seem likely to bring it into a far more important rank in the divisions of the city than it has heretofore held.

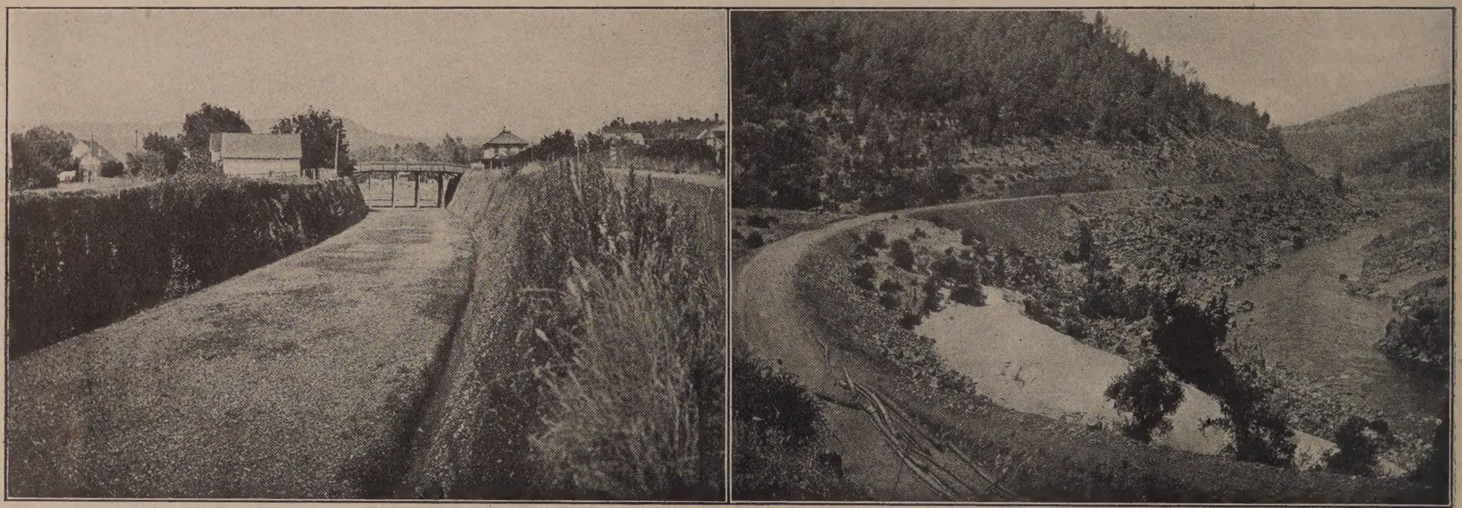
The Break in Copper Prices is one of the most interesting recent commercial incidents to engineers, for it brings down to a reasonable basis of cost a very important metal for many industries. While the manifold extensive uses of it in electricity have increased surprisingly in the last decade, the production has also increased, and it has seemed very strange for some years that the price of the metal should remain so high. The explanations advanced, asserting enormous demand and barely sufficient production, hardly seemed consistent with known facts concerning metallurgical operations while it was felt that market manipulation was an important factor in the conditions. It is now known that these surmises were correct. There is an enormous stock of unsold copper on hand which has at last caused a sensational drop in prices, and the high prices for the metal have resulted in the production of alloys which may be substituted for it with a saving in cost for equal efficiency. The composite cable of steel and aluminum wire recently introduced for transmission lines is an example, and the substitution of aluminum for copper in various parts of electrical apparatus is going forward steadily. Nevertheless copper must always have an important place in the industrial world, and for this reason a study of the conditions leading to the present situation is advisable as likely to suggest a way to avoid another abnormal upward soaring of prices under the manipulation of masters of high finance.

The International Photometric Commission which met at Zurich this summer has settled some things regarding primary standards of light now in use. There are three of these, the hefner, the pentane standard and the carcel. The last is deficient in uniformity and reproducibility, and is not as good as the others; its importance is due rather to its considerable use than to its excellence. The hefner is open to criticism on account of its small power and inconvenient color, and the 10-c.-p. pentane standard is relatively complicated and subject to variations with atmospheric fluctuations. Moreover, in spite of the skill shown in experiments to compare the luminosity of the various standards there is an outstanding discrepancy of 5 per cent. in the results to be accounted for in some way. The discussion by the Commission shows that the investigation leading to a satisfactory primary standard of light will be a long one, since not only has the thing itself to be found, but the inertia of long usage must be overcome.

# ENGINEERING FEATURES OF THE WESTERN PACIFIC RAILWAY.

By George P. Low.

When the Central Pacific R. R. was built, the aim was to secure railway communication between Ogden and the Pacific Coast by the easiest and most feasible route, to obtain which heavy subsidies were granted by the national government. The problem then was one of construction, viewed in the light of which its builders accomplished a marvelous piece of engineering work. To-day the problem to be solved in building a transcontinental railroad pertains to operation rather than to construction. The distinction is marked; where the Central Pacific spanned the distance from Ogden to San Francisco by the most readily-constructed route, the Western Pacific, in practically paralleling it, as shown in the map, virtually ignored any and all difficulties of construction where operative efficiency could be increased by so doing. In consequence, the Western Pacific engineers are to-day hewing a grade out of the granites of the Sierra Nevada Mountains that in many respects constitutes the most remarkable piece of heavy railway construction in existence.



The Western Pacific Railway Grade through Oroville and Five Miles East of That City.

The Western Pacific line will be longer than that of the Central Pacific by 143 miles, but its maximum grade after having been compensated for curvature will not exceed an actual 1 per cent., while that of the Central Pacific is 2.6 per cent (not 2.2 per cent. as claimed), with many miles of 10 deg. curves which bring the maximum gradient up to virtually 3 per cent. On the Western Pacific the maximum curvature is 8 deg. The Central Pacific crosses the Siera Nevadas at an elevation of 7,019 ft. above sea level, which necessitates the expensive maintenance of some 40 miles of snowsheds. The Western Pacific crosses these mountains at an elevation almost exactly 2,000 ft. lower and no snowsheds are necessary; during last winter, which was unusually severe, the maximum depth of snow on the line of the Western Pacific was 4 ft., while on the Central Pacific it was in excess of 40 ft.

It is anomalous that a piece of engineering work which is as heavy and as remarkable as is the trans-Sierran line of the Western Pacific should be so scantily possessed with features of engineering interest—indeed, the line presents greater opportunities for depicting natural scenery than for describing engineering achievement. The work is heavy in that practically throughout the mountains the grade is cut into the sides of rock-walled river and creek canyons, and it is remarkable, not only in that it crosses the Sierras at a comparatively low altitude, but in the manner in which it gets added distance and a lighter grade than would have been possible by adopting the more direct route presented in the canyon of the Middle Fork of the Feather River. The Sierra Division, covering 163 miles between Deep Hole, Nev., and Spanish Creek Crossing, Cal., contains two tunnels and a loop worthy of particular mention, and the North Fork Division, of 76 miles between Spanish Creek Crossing and Oroville, contains some special structures, but aside from these few features, standardization in all work prevails so universally that to describe one tunnel, for instance, describes all.

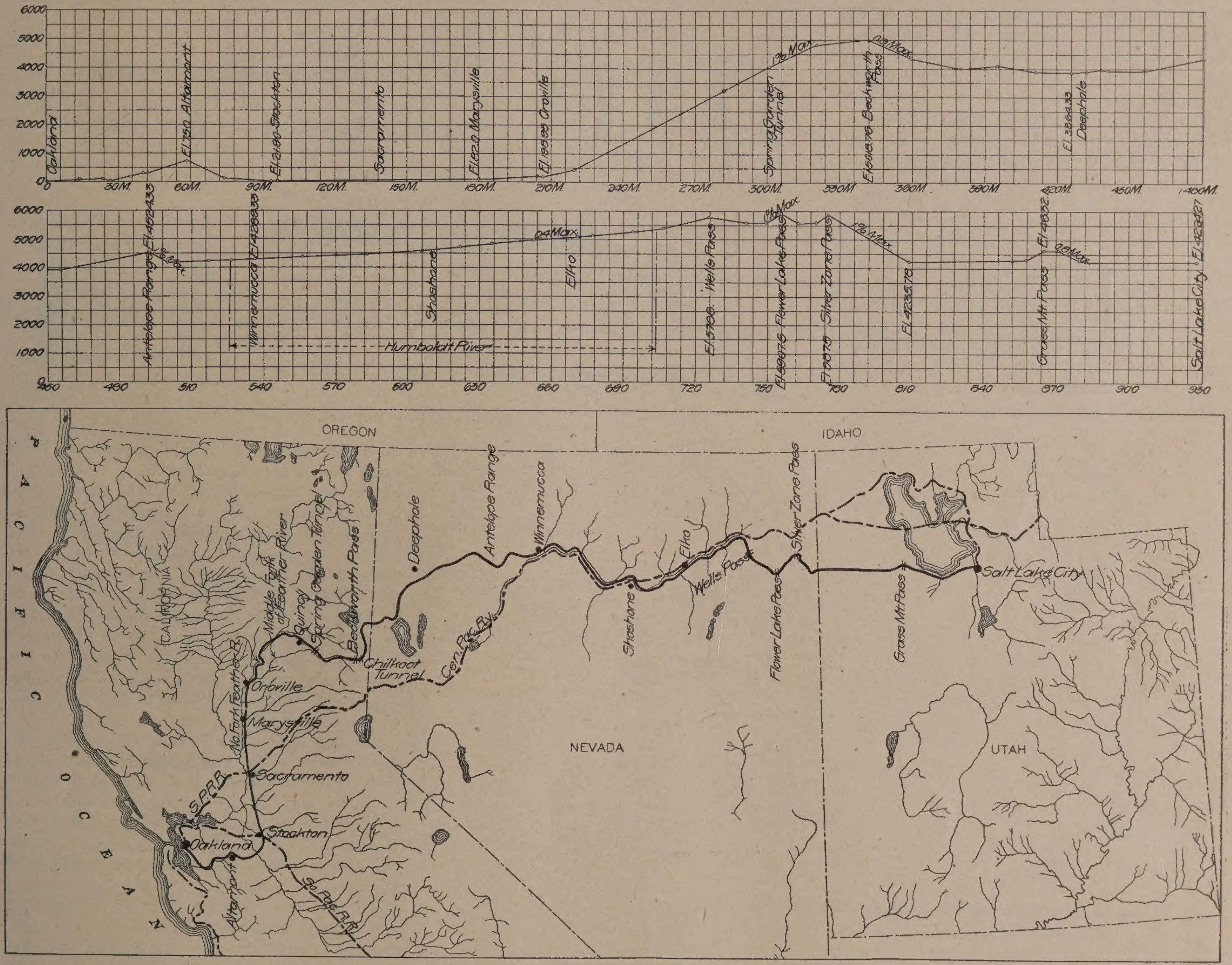
The most interesting portion of the line is in California from Chilkoot Tunnel, near the Nevada State line, to Oroville through a practically inaccessible region, although in the early days Beckworth Pass, which the line occupies, was well traversed. In traveling westerly, the Nevada-California State line is crossed at an elevation of 4,359 ft., whence the grade turns sharply to the south and enters Lake Valley, along which it parallels the Nevada, California & Oregon R. R. for about 17 miles, when it again takes the westerly course and, rising constantly with a maximum grade of 0.8 per cent., crosses the main divide of the Sierra Nevadas through the Chilkoot or Beckworth tunnel at an elevation of 5,018.76 ft. above sea level, the elevation of the trail over Beckworth Pass proper being about 5,250 ft. A slightly descending grade (0.4 per cent.) takes the line some 20 miles before reaching the town of Beckwith, and the head of the maximum grade of 1 per cent. extending practically all the way to Oroville, which is at an elevation of 199 ft.

The Middle Fork of the Feather River is reached within half a mile of the town of Beckwith, three crossings being made during the ensuing 12 miles. For 30 miles the grade follows the Middle Fork in a general northwesterly trend until reaching Spring Garden Tunnel, where it leaves the river at a point 15 ft. above extreme high water mark and cuts northwesterly through a divide which brings the line out on a small tributary of the North Fork of the Feather River. Though 80 miles of land lying within the State of California has been traversed thus far, the country, with its sage brush and cacti and arid wastes of sand, is characteristic of Nevada rather than of the Golden State, but on emerging from Spring Garden Tunnel, the westbound traveler will enter into a scene of paradise, so strongly contrasted are the Eastern and Western slopes of the Sierras.

It would have shortened the distance from Spring Garden Tunnel to Oroville by approximately 45 miles to have continued down the canyon of the Middle Fork to Oroville instead of leaving it for the North Fork of the Feather, but by so doing the grade would have dropped from 4,000 ft., making an inadmissible gradient of considerably more than 1½ per cent. Besides the slopes of the route selected, though extremely difficult, are more practicable to build over than those of the Middle Fork. Within a mile after leaving the lower portal of Spring Garden Tunnel, the grade enters Williams Loop, which has a difference in elevation of 33 ft. between the two tracks where they cross. The loop has a maximum diameter of 1,600 ft., crosses Spring Garden Creek twice and Pine Creek once, its maximum curvature is 8 deg., and its minimum 5 deg., and its length is about 5,600 ft. Seven miles below the loop, the grade leave Spring Garden Creek for Spanish Creek, which it follows until reaching, first, the East Branch of the North Fork and then the North Fork of the Feather River, when its general trend becomes southwesterly in a course that is maintained along the canyon of the North Fork until reaching the junction of the North and Middle Forks which occurs at a point about 7½ miles from Oroville. Thence the canyon of the main river is followed until Oroville is reached.

Before determining upon the final location of the grade, a careful reconnaissance was made of the river canyons to be traversed, during which the extreme high water line was accurately determined, for of all destructive agencies to which engineering works in the Sierra Nevadas are exposed, water is recognized as the one whose action is the most sudden and violent, and which, if not anticipated and fortified against, is almost invariably attended with the gravest consequences. Seldom does the constructing engineer meet with altogether unexpected problems of more serious import than those which arise from the torrents of these most erratic streams which perennially discredit the predictions of "oldest inhabitant" wiseacres by out-doing all previous performances. During this reconnaissance the extreme high water mark—that of 1881—was clearly delineated along the canyon sides. At the junction of the North and Middle Forks, for instance, it was at an elevation of 275 ft. above sea level. Early in March last, a flood of unusual severity occurred during which the water exceeded by a few feet the record established by the flood of 1881. It was then believed that the country would be immune from excessive flood for years to come, but the storm in the mountains was renewed with redoubled vigor and on March 18, the level of the river at the junction of the North and Middle Forks reached an altitude of 293 ft., or approximately 18 ft. above the high water of 1881, while the crest of the river at Pier 2 of the Western Pacific bridge across the Middle Fork reached 297 ft. above sea level.

A few incidents will illustrate the deceptiveness of these Sierran streams. The Great Western Power Co. has begun the construction of a series of hydro-electric and steam power generating plants in Central California which are to operate high-tension, long-distance power transmission lines that will commercially parallel the existing systems of the California Gas & Electric Corporation. The principal hydro-electric generating plant of this company is to be located at the Big Bend of the North Fork of the Feather River, 27 miles above Oroville. Buildings were erected last winter by the Great Western Power Co. to house the forces which were to have been engaged in the construction of the power plant during the present summer, but the flood of March last washed away the buildings and compelled the abandonment of all plans made for the location of the plant at that particular site.



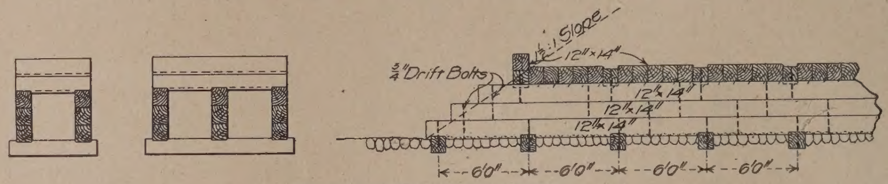
Map and Profile of the Western Pacific Railway.

An assistant engineer of the Western Pacific Ry. had his headquarters at the junction of the North and Middle Forks of the Feather River. The office building was at the top of a hill, and a tent wherein lived the engineer and his wife was not far away. Perfect security in its location was felt after the first flood of March had receded, but on March 18, most unexpectedly the waters of the river were awash the floor of the tent, which shortly went downward with the flood.

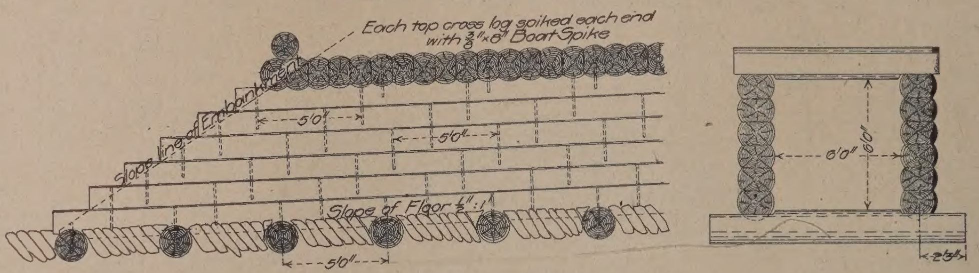
Eight miles below, at Oroville, the streets were flooded from 5 to 6 ft., buildings were floated from their foundations and wrecked, the bridge across the Feather River to Thermalito was washed away, and one of the largest gold dredges of the Oroville district was torn from its moorings and utterly demolished. For the first time in its history, Oroville has become convinced that it must be protected by levees. Were it not for the fact that other cities in Central California are leveed, the cosequences of the flood elsewhere would have been equally as disastrous as at Oroville. At Marysville, for instance, the wagon bridge across the Yuba River was washed away, and the new steel bridge of the Southern Pacific Co. near by was practically under water. The track of the Northern Electric Co., operating a third rail electric system between Oroville, Chico and Marysville, was inundated for several miles, and the grade of the Southern Pacific Co. was washed out in some places and the damage wrought to its trestles, culverts and roadbed was so extensive that its monetary loss thereby probably reached $1,500,000.

These matters have been gone into thus fully in order to point out the fact that the occurrence of the unprecedented floods of last March was not only confirmatory of the sound engineering judgment of the locators of the Western Pacific Ry., but was providential as well in that it indicated the necessity of so building the roadbed that its various structures should be of the most stable character. As a result of the experience, nearly all wooden trestles have been abandoned in favor of earth fills, concrete masonry and steel structures. Aside from last winter's lessons, however, the fact remains that with the exception of a few landslides, the removal of which would not have caused an hour's interruption in the operation of the finished line, absolutely no damage was done to those portions of the Western Pacific roadbed which were completed, for it was fully realized at the outset that every precaution must be taken to protect the roadbed against the action of water and to carry the water away from it. In fact, the late floods proved that the waterways and culverts were too large rather than too small.

In no case were the engineers misled by apparently dry gullies, as their lengths and drainage areas were fully investigated and ample waterways were provided for them before construction was begun. When, during the progress of the work, severe storms occurred, the assistant and resident engineers went over the work, observing closely the quantity and action of the storm waters with the view of determining more definitely the sufficiency of the openings contemplated or the advisability of reducing the size determined upon. Ample surface ditches were provided to carry off the storm water to the culverts and openings and to keep the water from saturating the slopes of cuts or undermining the foot of embankments, causing them to slide. Where a ravine was found to descend abruptly to the roadbed, a catch basin was excavated to hold in every case at least 5 cu. yd. of debris in order to prevent the culvert from becoming choked. All culverts were built so that they would discharge water under any head that could possibly arise without injury to themselves or the embankment. Whenever practicable they were laid out at right angles to the center line and set low enough to drain the borrow pits. All culverts were built in solid ground wherever posssible, but where it was found impossible to secure such foundation, the bottom was planked to prevent leakage and crowned slightly in the center to allow for settlement. Care was exercised in every case in filling around culverts not to tip or shove them out of place, inclined braces, both inside and outside the culvert, being used to prevent this. Blind drains, being regarded as objectionable, were always avoided unless the amount of water to be disposed of was known to be very small.

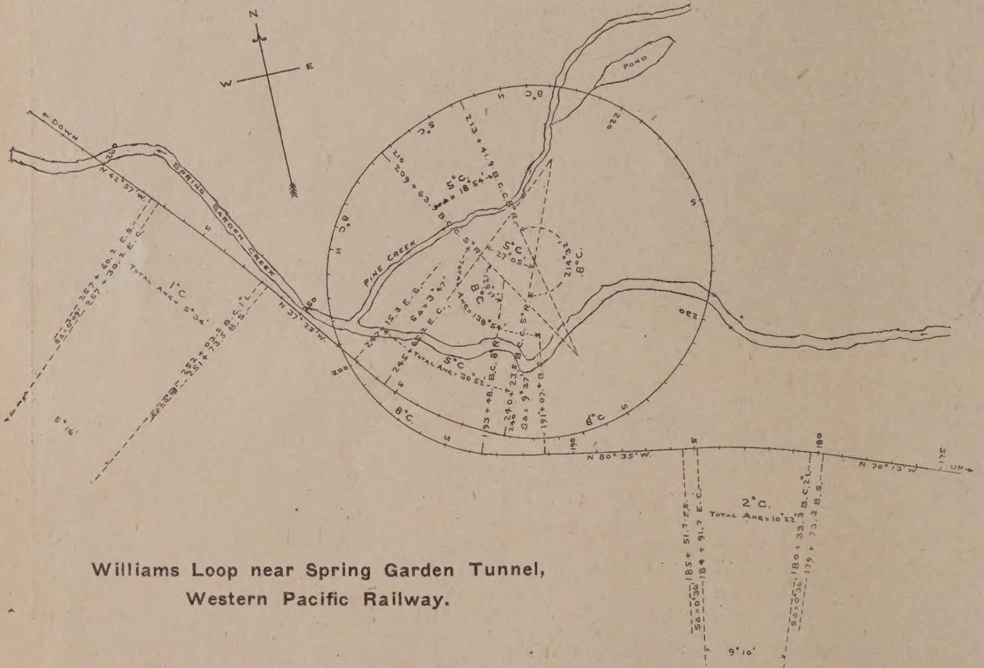


Standard Timber Culvert.



Standard Log Culvert.

There are 44 tunnels in the line of the Western Pacific between Oakland and Deep Hole, Nev., of which three are in the Altamont hills east of Oakland, thirty are in the North Fork Division, and eleven are in the Sierra Division. The Chilkoot Tunnel, which is 6,000 ft. long and as stated pierces Beckworth Pass, is on a tangent with no engineering features of interest regarding location. It is of interest, however, in that it is east of the heavy snow fall region of the Sierra Nevadas. Usually the Pass is bare of snow; seldom does more than 1 ft. of snow fall upon it, and the heaviest snowfall ever known occurred in March last when it covered the ground to a depth of 2 ft. Spring Garden Tunnel, 7,400 ft. in length, is the longest tunnel in the line, but its interest centers in the advantage it gives of added distance and a lighter grade, as previously explained. The total length of tunnels in the North Fork Division aggregates 12,438 ft., the longest tunnel being 1,220 ft. and the shortest, 150 ft.



Williams Loop near Spring Garden Tunnel, Western Pacific Railway.

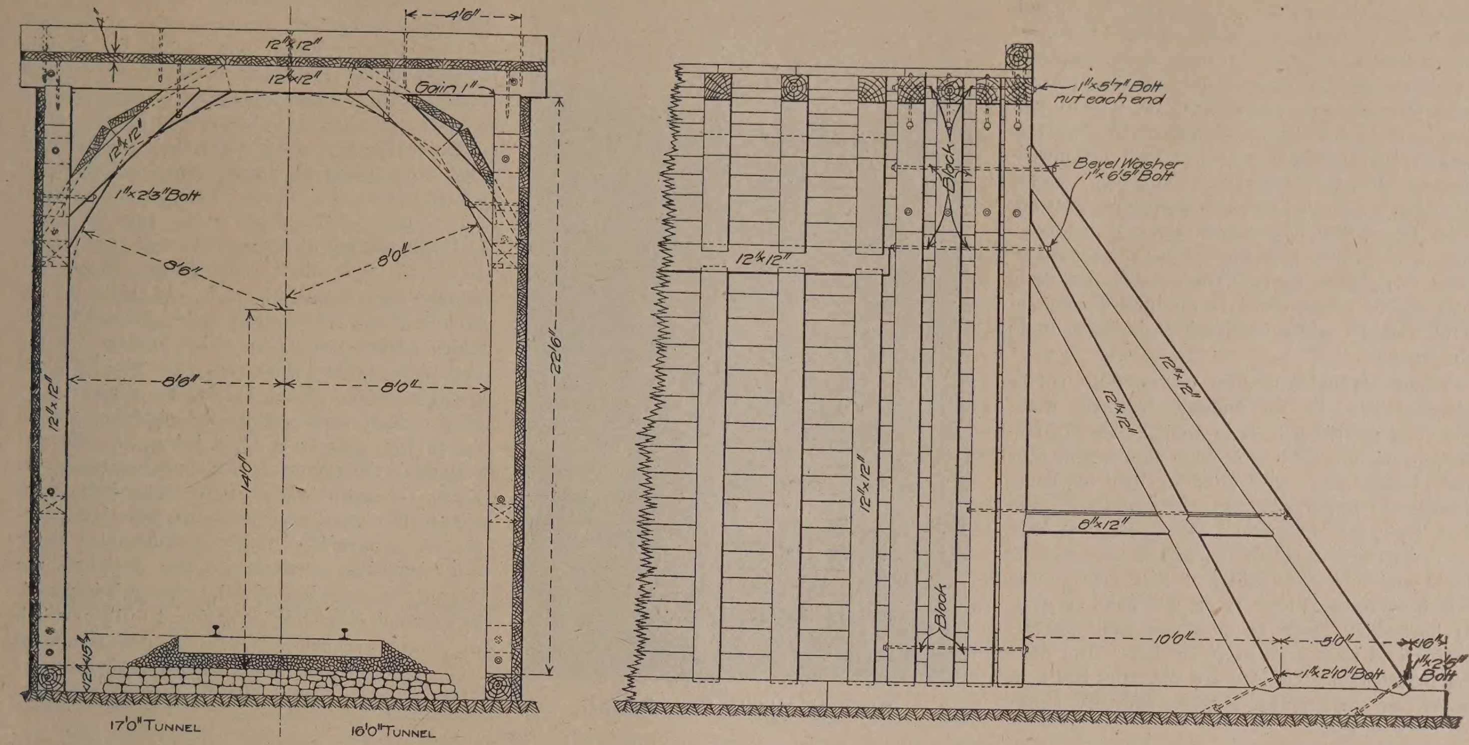
All tunnels in the Sierra Nevadas are being taken out for single-track construction, their normal cross section being not less than 16 ft. wide nor more than 17 ft. wide between vertical side walls, 22½ ft. high above sub-grade, and 1 ft. below sub-grade. The standard 16-ft. section tunnel is being built on tangents and curves of 3 deg. or less, and on curves of over 3 deg., the 17-ft. section is used. It has been found as a general rule that in excavating the approach, the cost of tunnel and of cut about balance when a depth of 50 ft. on the center line of cut has been reached, hence if solid material is encountered, the heading is placed approximately on this center line, but if the excavated material is loose and difficult to hold, the heading is started as soon as sufficient depth to secure a good face has been reached, as in so doing the weight of the heading timbers is reduced to minimum. The curves of the arches of these tunnels are a semicircle whose springing lines are 14 ft. and 14 ft. 6 in. above sub-grade in the 16-ft. and 17-ft. tunnels respectively. The normal cross-section for single-track tunnels contains an area of 368.4 sq. ft., equaling 13.65 cu. yd. per lineal foot of tunnel.

Tunnel 4, which pierces Cape Horn, on the Feather River about 2½ miles east of Oroville, may be accepted as typical of all tunnels on the North Fork and Sierra Divisions. It is a standard 17-ft. tunnel, 806 ft. long and built on a curvature of 8 deg. Work on the easterly and westerly approaches was begun in November, 1905, and in January, 1906, respectively, and the excavation of the heading was completed in October, 1906, while that of the bench was finished in January last. During the first one or two months, considerable progress was made in excavating the approaches, which were of loose rock and conglomerate, then solid rock was encountered and as hand drilling only was used, the progress was slow until February, 1906, when compressed air became available at the east portal. The advance was more rapid thereafter until in May a stratum of exceedingly hard rock was encountered so that during May and July but approximately 15 ft. headway was made. Then the material became easier to work and in November the maximum cut of 95 ft. was made in the bench. A similar condition existed from the west end where compressed air was introduced in March. The heading was finished in October and the bench in January last, the tunnel being completed a few weeks later.

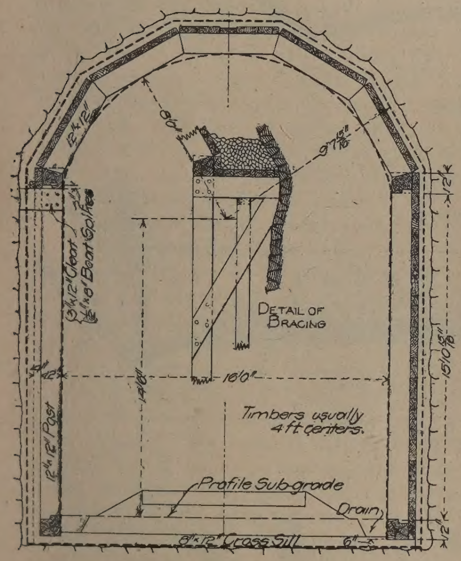
The portals and lining are of Oregon pine, their main timbers being 12 x 12 in. in size. At each portal, eight upright posts, lagged together with 6x12x12-in. blocks, bolted and toe-nailed, rest on 24-ft. sills and are stiffened by two outer and two inner main braces each 12 x 12 in. and 22 and 26 ft. long respectively, the posts being capped by 22-ft. timbers which are lagged with 4x12-in. planking continuing throughout the tunnel. The bents are 12 x 12 in. in size, set on 3-ft. centers with 4x12-in. plank lagging. In framing timbers the joints at the tops of segments were made to flare a little so that when pressure came on them, they were brought to a close fit throughout the flare. Particular attention is given to see that the dry packing used is tamped in solidly, an inspector being employed to look after this work.

Roadbed sections are in three forms, earth filling for single track, and ballast, broken stone or cinder filling for double and single track respectively. From Oroville through to Stockton the track will be ballasted mainly with the gravel tailings of the gold dredgers in and about Oroville, and considerable of the track of the North Fork Division is to be similarly ballasted. Earth filling is to be used very sparingly, if at all, through the mountains, but when used it is to be made highest in the center where, for one foot on each side of the center line, it will be 3 in. deep over the top of cross ties, thence sloping off each way to the bottom ends of the ties. The ends of all ties are to be left just clear of the ground, and the roadbed outside and beyond the ends of the ties is to have a uniform slope descending away from them.

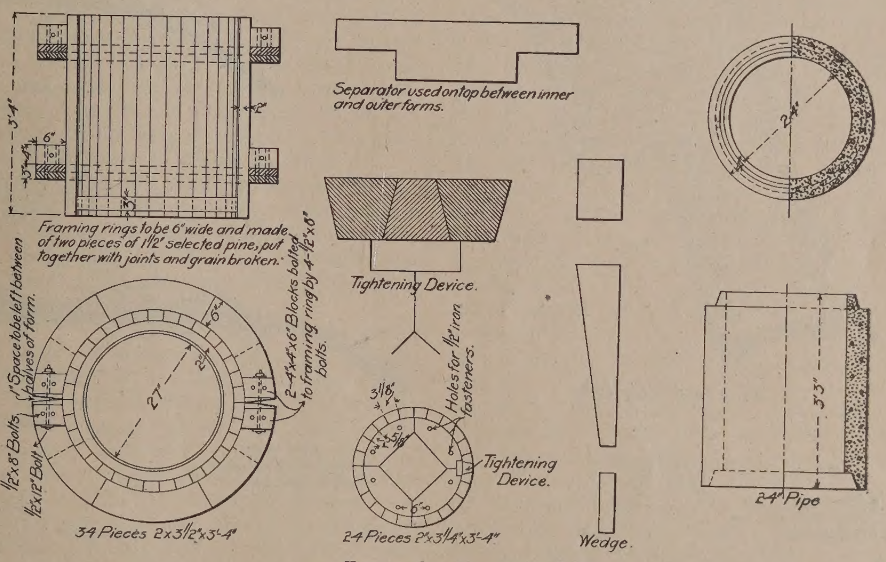
Retaining walls of either dry rubble or concrete or masonry are used in preference to log cribs wherever rock that breaks out in slabs can be secured. Their foundations are always sunk to a solid footing, the face batter varying accordingly to the type and height of the wall. The limiting heights of dry rubble and concrete or masonry walls are 20 and 30 ft. respectively. The stones used in building rubble walls vary in size and material with the character of the excavation, quarries or borrow pits whence they are obtained, but no stone used on the face of the wall is less than 8 in. in thickness nor less than 18 in. in its least horizontal dimension. Headers are at least 3 ft. long, unless the wall is 3 ft. in thick-



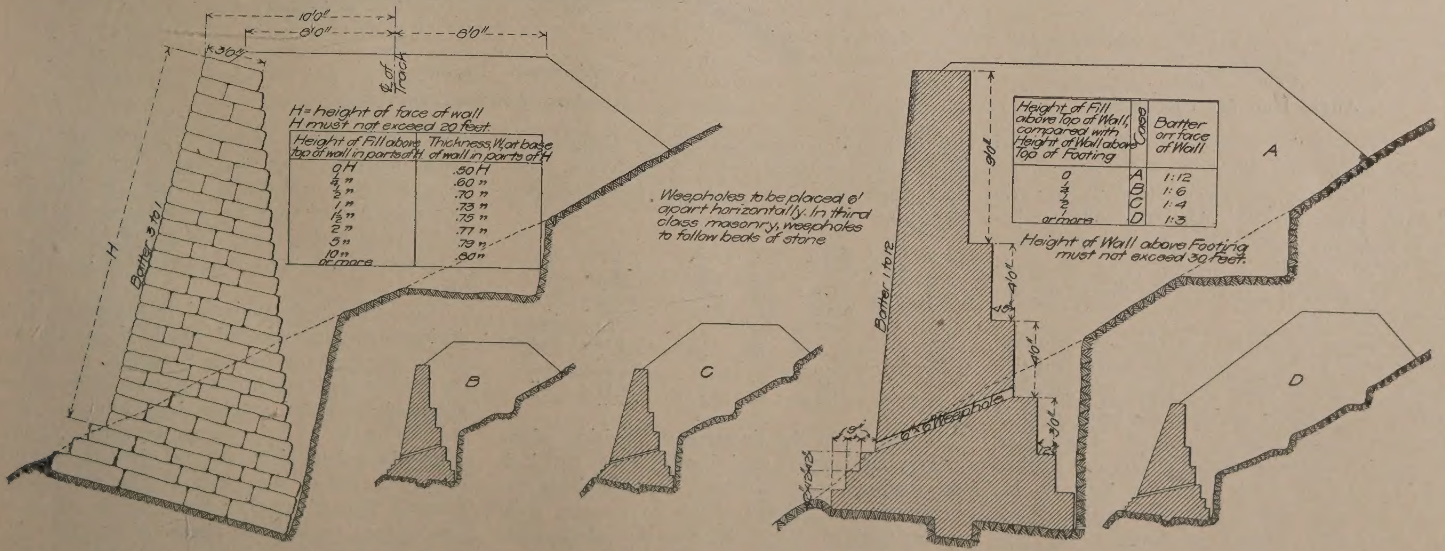
Portal for Timbered Tunnel.



Timbering in Medium Rock.


3'4"
6"
2"
3"
Framing rings to be 6" wide and made of two pieces of 1½" selected pine, put together with joints and grain broken.
Separator used on top between inner and outer forms.
Tightening Device.
1" Space to be left between halves of form.
2-4"x4"x6" Blocks bolted to framing ring by 4-½"x8" bolts.
½"x8" Bolts.
½"x12" Bolt
27"
24"
6"
34 Pieces 2x3½"x3'-4"
3⅛"
Holes for ½" iron fasteners.
Tightening Device.
24 Pieces 2"x3¼"x3'-4"
Wedge.
24"
3'3"
24" Pipe

Forms for Concrete Pipe.


10'0"
8'0"
8'0"
3'6"
℄ of Track
H
Batter 3 to 1
Weepholes to be placed 6' apart horizontally. In third class masonry, weepholes to follow beds of stone
B
C
A
9'0"
4'0"
15"
4'0"
3'0"
Batter 1 to 12
6" tile weephole
D

H = height of face of wall
H must not exceed 20 feet.

| Height of Fill above top of wall in parts of H. | Thickness, W, at base of wall in parts of H |
|---|---|
| 0 H | .50 H |
| ¼ " | .60 " |
| ½ " | .70 " |
| 1 " | .73 " |
| 1½ " | .75 " |
| 2 " | .77 " |
| 5 " | .79 " |
| 10 " or more | .80 " |

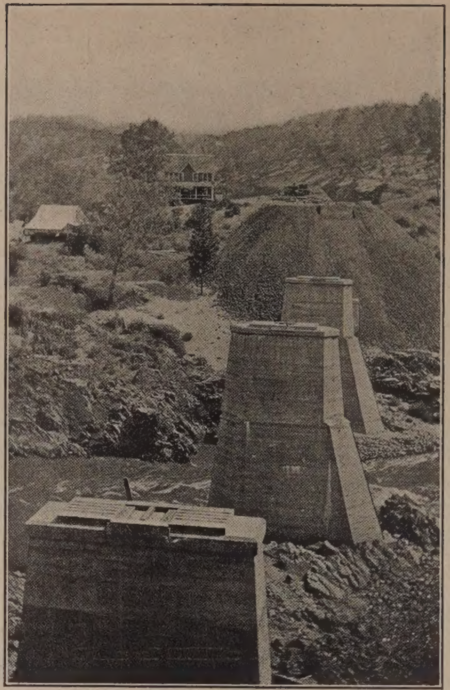
| Height of Fill above Top of Wall, compared with Height of Wall above Top of Footing | Case | Batter on face of Wall |
|---|---|---|
| 0 | A | 1:12 |
| ¼ | B | 1:6 |
| ½ | C | 1:4 |
| 1 or more | D | 1:3 |

Height of Wall above Footing must not exceed 30 Feet.

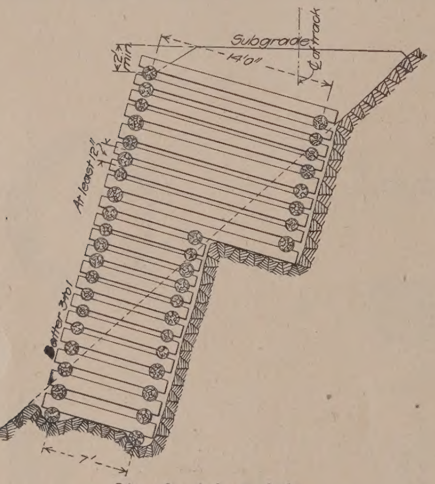
Standard Types of Masonry and Concrete Retaining Walls.

ness, in which event they extend entirely through the wall, at least one header being placed for every three stretchers. On steep slopes, steps are cut in the natural slope to prevent the embankment from sliding, regardless of the type of wall used. Walls built near the line of the roadway and having a height in excess of 10 ft. are invariably of third-class masonry or concrete. Ample drainage is provided through weep holes that are 6 in. square and placed 6 ft. apart as nearly in line as is possible with the natural slope, particularly if it be of stone. These walls rest on a base which is excavated to an inward slope of 1:12, and the center of this base is keyed in to the footing.

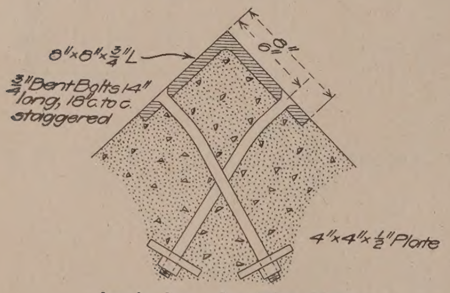
Where a suitable quantity of stone can not be obtained near by for building retaining walls, log cribs are being built to hold the foot of embankments, provided their maximum height does not exceed 30 ft. Such cribs are built log-house fashion, of round logs with bark removed. The face and rear longitudinal logs average at least 14 in. in thickness at the butts, and are laid with butts and tops alternating, so that each course, which averages at least 12 in., will build up nearly horizontal. Both tie logs and wall logs are notched at least 2 in. deep, so that a firm bearing will obtain. Tie logs are generally laid 8 in. apart, and the structure is bolted together, log by log, with ⅜x18-in. drift bolts. The batter of the face varies with the slope of the ground, but generally it is 1:4, and the width of the log crib similarly varies with the slope, the endeavor being to make the width of its base as great as possible up to one-half its height. The sub-grade is at a minimum of 2 ft. above the line of the uppermost face log.



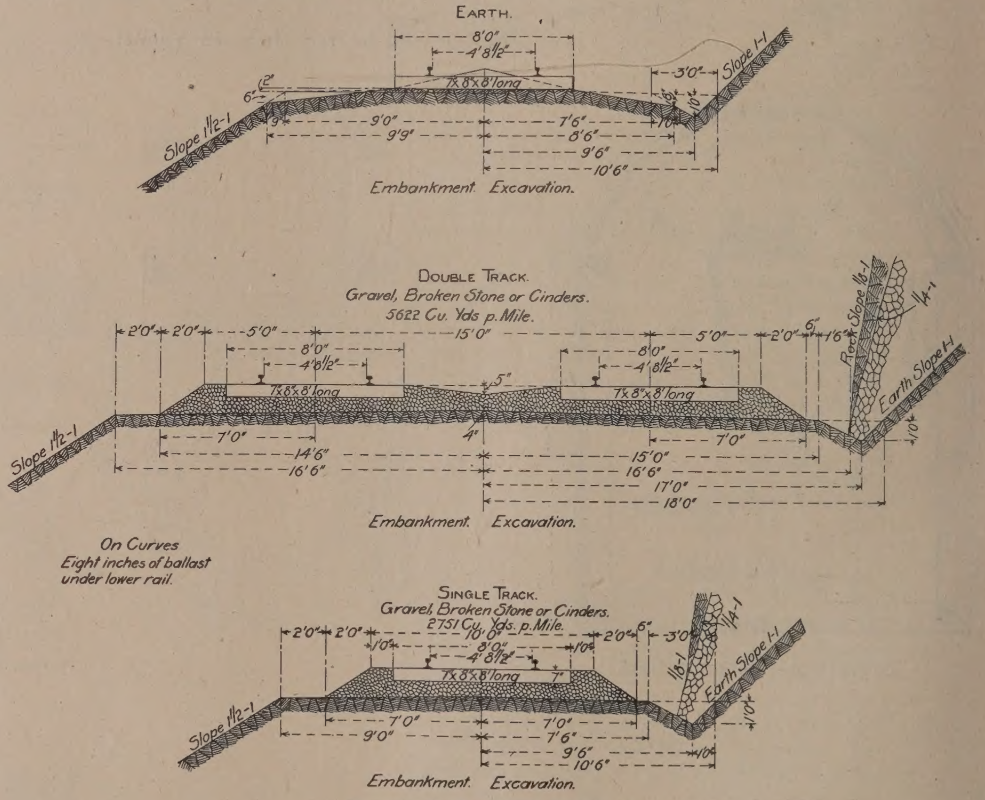
Piers of Middle Fork Bridge.



Standard Log Crib.



Angle Iron for Pier Nose.



Standard Roadbed Sections.

In drainage openings of a size not requiring arch, timber or log culverts, or where suitable material is not at hand for stone box culverts, concrete pipe culvert openings are being used in diameters varying from 12 to 48 in. These pipes are made of washed gravel or quarry-run crushed rock, no axis of which is greater than 1½ in., mixed wet with cement in the proportion of 1:4 and thoroughly tamped in forms. The pipe lengths are jointed with neat cement after having been laid on lines and firmly bedded. All pipes have a uniform length of 39 in. and a uniform depth of 3 in. at each end, their remaining dimensions varying as follows:

| Inside Diam., in. | Thickness of Main Shell, in. | Thickness of Shell at Tips, in. | Base of Taper, in. | Width of Shoulder, in. | Cost of Complete Form. | Pattern. |
|---|---|---|---|---|---|---|
| 12 | 2 | ¾ | 1¼ | .. | ..... | Bell |
| 12 | 3 | 1 | ½ | 1½ | $13.75 | Straight |
| 18 | 3 | 1 | ½ | 1½ | ..... | " |
| 24 | 4 | 1½ | ½ | 2 | 15.75 | " |
| 36 | 6 | 2½ | ½ | 3 | 22.75 | " |
| 48 | 8 | 3½ | ½ | 4 | ..... | " |

In places on the Sacramento Division where culvert openings are used through levees on which the roadway runs, or where, in maintaining the grade, the embankment tends to restrain flood waters to their natural channels, the culvert pipes are provided with swinging check valves.

The bridges of the North Fork Division are to be nine in number, two across the North Fork of the Feather River, one across the Middle Fork, and the remainder being across various creeks. All of these bridges of consequence are to be of steel, and vary in length from a 60-ft. plate girder bridge to a 480-ft. plate girder and riveted deck span bridge at Spanish Creek. Bridge 18 which spans the Middle Fork at its junction with the North Fork is a typical struc- and will support two 125-ft. riveted deck spans and three 64-ft. deck plate girders, giving the bridge a total length of 442 ft. At this point the Middle Fork courses over a bed of solid rock which affords thoroughly stable footings for the four piers and two abutments. The tops of Piers 2 and 3 are on a level, barring the slight rise in grade which exists, while the elevation of the top of Pier 4 is some 13 ft. below, and the elevations of the abutments are approximately 20 ft. above, the mean height of the tops of the main piers. The piers and abutments were built during an extreme low water period which made cofferdamming unnecessary. The structures are of concrete, 2,230 bbl. of Atlas cement being used, its economic efficiency being found to be 1.07 bbl. of cement per cubic yard of finished structure. The mixture used was in the proportion of 1:2½:5, with considerable large rock incorporated in the concrete of each structure, great care being taken to clean thoroughly and wet each stone before being placed. They are laid not closer than 18 in. from any surface, nor less than 24 in. from bottoms, and are placed at least 12 in. apart so as to give sufficient room for tamping.

These piers were subjected to a supreme test during the unprecedented flood of March 18, when, as already stated, the torrential waters of the Middle Fork reached a mean of 18 in. above previously recorded extreme high water as discernible by the wash in the canyon sides. At the point where these piers cross the river, the stream showed a crest of about 4 ft. at Pier 2, reaching within a foot of its top. Some 600 ft. above, a wagon road crossed the chasm of the river by means of a combination steel and timber bridge having a 140-ft. span, which was lifted off its abutments and carried down stream transverse to the axis of the river. The center of the runaway bridge struck Pier 2 with terrific violence. For a moment the bridge quavered before breaking at its center, and, being held by the pier, as its ends swung together the bridge was lifted over the top of the pier by the backwater created, and then carried on down stream. Despite the fact that the forms had not been off the pier thirty days, it was absolutely uninjured save for a scratch on its nose.

The engineering organization of the Western Pacific Ry. Co. was explained in this journal on Aug. 3 of this year. The work is under the direction of Mr. Virgil G. Bogue, first vice-president and chief engineer. Mr. H. M. McCartney is the principal assistant engineer, with offices at San Francisco, and the division engineers are Messrs. C. Harlowe at Cobre, Nev., J. Q. Jamieson at Clio, Cal.; L. H. Jones, at San Francisco; Emery Oliver at Oroville; J. T. Williams at Oakland, and T. J. Wyche at Salt Lake City.

## Calf Killer Bridge, Nashville, Chattanooga & St. Louis Ry.

The section of the Sparta branch of the Nashville, Chattanooga & St. Louis Ry. from Sparta to Bon Air was built in 1887, the crossing of the Calf Killer River being part of the work. At the south edge of Sparta the river makes a bend of 160 deg. inside of 700 ft., and the bridge was built in this bend on a tangent at right angles to the flow. The grade of the bridge was 1 per cent., ascending toward Bon Air. About 500 ft. south of the bridge the grade changed to 3 per cent, and, with the exception of four short benches, continued ascending at that grade to Bon Air. This put the Calf Killer Bridge at the base of eight miles of steep grade.

The masonry for the bridge consisted of two end piers, one middle pier and a stem wall at each end. The first pier (the north pier) was founded on concrete at a depth of 4 ft. below the surface of the natural ground; the second pier was founded on solid rock in the bed of the river at a depth of 48 ft. below base of rail, and the third pier (the south pier) was founded on solid rock at a depth of 45 ft. below base of rail. The distance from base of rail to bridge seat was 25 ft. 1⅛ in. The masonry for these two spans was made to take Phœnix-column deck trusses of Warren type, removed from the Tennessee River Bridge at Bridgeport, Ala. Their length, center to center of end pins, was 153 ft. 4 in., and width, truss to truss centers, 12 ft. There were five panels to the truss, of length 30 ft. 8 in. This bridge had been in service about fifteen years previous to its re-erection at Calf Killer in 1888.

The principal reasons for putting up a new bridge may be briefly stated thus: The old bridge was too light for present-day traffic; its location at the foot of the long, steep grade made it more dangerous than usual for trains of lost control coming down the mountain; the top chords of the trusses being of cast iron, made their strength an uncertain factor, and the style of columns and bases were such that a serious defect might be present and pass undetected. For the renewal of the entire deck of this old bridge 2-ply 8x16-in x 32-ft. wooden chords on the top chords of trusses on each side for the full length of the bridge and 8x14-in. x 14½-ft. ties were required. The deck of the new bridge requires only the 8x9-in. x 10-ft. ties.

A number of plans were suggested for rebuilding the bridge. One was to fill one-half of the north span and put a girder on the other half and replace the south span with a stronger span from Bridgeport; estimated cost, $23,000. Another was to renew the two spans on the same masonry; estimated cost, $21,000. Another scheme was to use one 30-ft. girder and two 60-ft. girders supported on towers in place of the north span and replace the south span with a stronger span; estimated cost, $17,000. Another scheme was to build two new piers and top out the old ones, using four spans of 75-ft. deck plate girders; estimated cost, $11,850. This latter plan was adopted.

A survey was then made to get the physical data of the crossing necessary to make plans for the new masonry and to get the exact position of the old bridge on the masonry, especially noting its position at points likely to interfere with the construction of the new masonry. This was done with a view to carrying up the new masonry without erecting false work before the time to put in the new girders and without delaying traffic.

The procedure in the construction of the masonry as adopted was: (*a*) Build up piers 1 and 5, encasing the end vertical posts and end braces, removing the latter when the old structure is taken down; (*b*) finish complete piers 2 and 4, leaving slots, through which the old chord bars can be withdrawn; (*c*) erect on both sides of pier 3 falsework towers sufficiently strong to support the ends of the new girders, after which the old spans will be taken down and the new superstructure put in place, when the old center pier (No. 3), without obstructions, will be built up. By this design the old masonry was to be built up to the finish for 7-ft. deck plate girders, using vertical faces and not exceeding the size of the old piers. The length of this top section on the old masonry was to be 14 ft. on each of the piers, and the design of the entirely new piers is in size and shape similar to the old mid-pier, with its new top section. This gives the set a uniformity of design and effect.

The first work to be done on the construction of the masonry was to put in a cofferdam for pier 4, the only one coming in the bed of the river, to be built entirely new. The cofferdam was started when water in the river was about 6 ft deep.

As piers 2 and 4 came midway of the center panel of the old trusses and the top of the new masonry being 8 ft. 1 in. from base of rail, they were built complete, without being obstructed by the middle diagonals of the old bridge, except to box around the bars of the lower chords, filling in the slots with concrete after the old bridge had been removed. A temporary pile bent was previously driven at each end of the bridge to permit of the removal of the end vertical posts of the old trusses, so that the top sections of the end piers could be built up complete, except for having to incase the bases of the end braces, which small open sections remaining could soon be built out even with the finished faces. These slots and open sections were not large enough to lessen the strength of the masonry perceptibly for carrying the bridge and trains. When all the piers but the middle one were ready for the new bridge, falsework towers were put up on each side of the middle pier and the new girders put in place, their ends at the middle pier resting on these towers just about 2 in. above the top elevation to be given the pier.

The plate girders are 75 ft. center to center of bearings, 77 ft. 6 in. out to out of girders, and spaced 8 ft. centers. The web plate is 84 in. deep by 7-16 in. thick. There are two flange angles 6x6x¾ in. for full length of girders on top and bottom. The top has four cover plates, each 14x⅜ in.; lengths, 7 ft., 34 ft., 49 ft., and one the full length of girder. The bottom cover plates are of the same size and number as the top ones, except that the longest one is only 54 ft. instead of the full length of girders. The rivet pitch used is 3 in., with alternate spacing.

Before placing the girders falsework bents were put up between the old trusses through one span to the height of the new masonry. Pony bents were erected on top of these to carry the track. The wooden chords on the top chords of the old trusses were shifted to these pony bents for stringers. The ties were replaced with shorter ones, allowing space between their ends and the old trusses to lower the girders to the main bents and new masonry. Before the girders were lowered the lateral rods and braces of the top of the old trusses had to be removed. The girders were handled from trucks on the bridge to place by a gallows from a traveler bearing upon the top chords of the old trusses. By these arrangements trains were carried on falsework for each span about three weeks, one span being put in before the other was put on falsework. If the stability of the old spans, with top lateral rods and braces removed, had not been so greatly impaired and the time required to shift and handle the long and large bridge ties had not been so long, these girders could have been put in without trains having to run on false work, except for the tower bents at the center pier and the bents at both ends of the bridge in place of the vertical posts removed. When these girders were in place and the old spans removed the middle pier was built up complete without obstructions. Then after the concrete was sufficiently set the tower bents carrying the ends of the girders were removed and the girders let to their final position on the pier.

The arrangements for handling the concrete material and for making and placing the concrete is worthy of description. The sand and aggregate, consisting of blast-furnace slag from South Pittsburg, Tenn., were unloaded from cars to platforms on a level with the top of rail about 100 ft. south from the south end of the bridge. The concrete mixer, a cubical form of 1-6 cu. yd. capacity, run by a gasolene engine, was located on a platform built up about 50 ft. south from the south end pier. A tank near the mixer to supply the water was elevated enough to get the desired head and was kept filled by a pump run by another gasolene engine located down on the river bank. The cement house was close by, a little to one side, between the mixer platform and slag pile. Wheelbarrows were used to deliver the slag and sand to the mixer. The mixer was so placed that it would dump onto the platform, and the concrete was then shoveled into the specially designed narrow-gauge car, which used one rail of the main track and an extra rail on the outside for its running track. A turnout for clearing passing trains was provided for at both ends of the bridge. The grade of the track over the bridge was descending from the mixer 1.05 per cent., so that with a little start the concrete car would roll alone down to points on the bridge; and only in returning empty to the mixer did the car have to be pushed by hand, although this was never for a distance of more than 400 ft. Over the piers on the bridge in the center of the concrete car's track, openings were sawed out to let the concrete pass to the forms below. To get the concrete into the forms with very little disturbance and fall there were used zigzag chutes, with arms about 10 ft. in length, which sections were removed as the level of the concrete in the forms came up close to the end. This chute was a convenience because its end alternated from one side to the other as the arms were removed in coming up. A swinging section could have been provided for the end to swing the concrete to the desired spot in the form; but the extra time required to swing it about, to detach from one section and attach it to the next, would have been much more than that required to shovel the concrete to points in the form not directly under the end of the chute.

The foundation work for this bridge was built up by the railway's masonry gangs. The bridge company furnishing the girders put them in. The work was commenced about June 20, 1905, and the bridge was finished complete about Dec. 1 of the same year.

The cost of the masonry is shown in an accompanying table. There were 460 cu. yd. of concrete the cost per yard for the entire work being $6.49.

Cost of Foundations and Piers.

| | | Cost per cu. yd. |
|---|---|---|
| Unloading material | $ 88.15 | $0.19 |
| Labor on forms and bins | 435.50 | 0.95 |
| Lumber | 472.00 | 1.03 |
| Cofferdam—excavation, lumber and labor | 293.20 | .... |
| Mixing and placing concrete | 428.00 | 2.79 |
| Cement, slag and sand | 854.90 | |
| Taking down forms and cleaning up | 211.40 | 0.46 |
| Engineering and supervision | 200.00 | 0.43 |
| Total | $2,983.15 | |

The new steel cost $10,082.80 erected, and deducting the scrap value of the old spans, $1,800, leaves the net cost for the steel as $8,282.80. This added to the cost of the masonry makes a total of $11,265.95, which is $584.05 below the preliminary estimate, made when the reconstruction of the bridge was first determined upon.

The weight in pounds of the metal of the two old spans was 186,860. The maximum safe engine load in tons of the old bridge was 84. The weight of the four new spans is 268,451 lb. The maximum safe engine load of the new bridge is two 141-ton engines coupled, followed by a uniform load of 4,500 lb. per foot, with allowance for impact as specified by the American Railway Engineering and Maintenance of Way Association. The cost of the new steel bridge erected per pound was 3¾ cents. This does not include the cost of the new deck, as this was charged to the renewal account. The steel work could not be duplicated for the above cost, for an error was made in the calculation of the weight of the bridge by the company making the bid. It is estimated that about $1,500 should have been added to the bid for the steel work to make the total cost come up to the cost of similar structures.

The work was described by Mr. J. G. Huff in a paper before the Engineering Association of the South, from which the data for these notes were taken.

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Explosions of Coal Dust have caused so much trouble in cement mills that managers of such works will probably be interested to learn of experiments which Prof. Phillips Bedson described at the recent meeting of the British Association. His first experiment was made by fitting glass tubes 1½ in. in diameter into opposite sides of a 9-in. cubical tin box provided with a hole in the top and in the bottom so that a gas flame could burn within it. The dust was prepared for the experiments by passing it through a sieve with 100 meshes to the inch, and was then placed in one of the glass tubes and blown into the box by air pressure. It was found that some kinds of dust would inflame under this condition, flour and powdered aluminum flashing up violently, but coal dust acted in an uneven manner. In order that the experiments might give more definite results, Professor Bedson used a larger box and increased the charge of dust to two grams or more, which was ignited electrically while suspended in the box. Various modifications of the apparatus were made to enable him to vary the experiments. It was found that ordinary coal dust was not directly inflammable, the most inflammable coals seeming to be those which contained considerable volatile gas. The amount of moisture had little apparent influence so long as the dust was not absolutely wet. The main thing shown by the experiments was that it is necessary to make extended investigations before any definite general knowledge of the nature of these explosions can be obtained.

## Examination of Water Purification Plant at Owensboro, Ky.

By Philip Burgess, Assoc. M. Am. S. C. E.
Special Assistant Engineer, Ohio State Board of Health.

In the issue of The Engineering Record for Aug. 31, 1907, is found an interesting description of the intake of the Water Works Company of Owensboro, Ky. As the writer had just completed an examination of this plant on Aug. 28, 1907, with particular reference to results obtained, it is thought that a report of this examination may be interesting as a conclusion to the above mentioned description. The Ohio State Board of Health has been so frequently called upon for advice relative to the installation of so-called natural filtration plants, particularly on the Ohio River, that it has been considered desirable to obtain all the information available regarding existing plants of this type. The plant at Owensboro, Ky., had been cited as an example of a successful infiltration plant, and this examination was undertaken to study its principal features and to determine the degree of purification obtained.

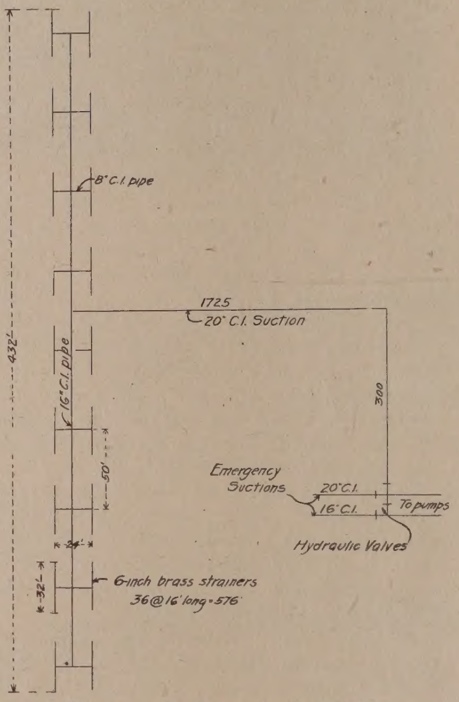

Diagram of Owensboro Intake.

Since May, 1906, the writer has been engaged by the Ohio State Board of Health in making examinations of the various water purification plants in Ohio, and the examination of the Owensboro plant was conducted in a similar manner to that in which these other tests have been made. Boxes containing the necessary laboratory media and supplies, especially built for this purpose, were shipped by express to the pumping station and such chemical and bacterial determinations as practicable made there, while samples of the water were shipped by express to the laboratory of the State Board of Health at Columbus for such determinations as it was not practicable to make in the field. Mr. Asa Williams, the chief engineer of the Owensboro Water Works Co., assisted the writer in making the examination of the plant and also furnished the data and descriptions embodied in this report.

Owensboro is a city of about 20,000 inhabitants, situated on the Ohio River about 115 miles below Louisville. Until recent years the water supply had been obtained directly from the river without purification and was unsatisfactory from all points of view, being very turbid at times and always subject to considerable pollution. About four years ago pressure was brought by the city officials on the Owensboro Water Co. to purify the supply. The water company was desirous, if possible, of installing some system in which the purification would not be dependent on the ordinary methods employed but one in which the necessary care and expense that are involved in the usual filter plants would be eliminated. After considerable consultation the present system was installed by which the purification is effected by the infiltration of water through the gravel bottom of the river and its collection into a system of horizontal brass strainers.

Several tests of the quantity and quality of the water obtainable were made before any construction was undertaken. Owensboro is fortunate in having a sand and gravel bar directly opposite the pumping station on the shore of an island, and it is from this bar that the present supply is obtained. In order to learn the quality of the water available, a small pipe was first driven into the gravel and water pumped from this by means of a hand-pump. In this way it was found that the supply was of satisfactory quality, but it was not possible to estimate the quantity obtainable. In a subsequent test a 2-ft. tile pipe was sunk into the bar to a depth of 8 ft., and the water which percolated into this well was pumped by a steam pump. It was noted that the water stood 58 in. deep in this well until the pumping was increased to a quantity greater than 50 gal. per minute. With a greater quantity than this, the level of water in the well was reduced, and finally at 90 gal. per minute, sand and water together were drawn up through the well. From these figures it was estimated that 50 gal. per minute was the maximum capacity of the well, corresponding to 16 gal. per square foot per minute. Water was pumped from this test well continuously for several hours and found to be of satisfactory quality, being clear and free from iron. At a depth of 14 ft. in the gravel, Mr. Williams stated that a water was obtained which soon turned red on being exposed to air and light and was evidently a ground water containing considerable iron.

The essential features of the plant as installed are a system of slotted brass pipes or strainers embedded in the bottom of the river at from 4 to 5 ft. in depth in the gravel bar, these strainers being directly connected by a system of collecting and suction pipes to the pumps by which the water entering the system is pumped into the mains without storage in a distributing reservoir. The river water thus percolates through the bed of the river into the collecting pipes, leaving the impurities behind on the sand and is not again exposed to the light until drawn from some part of the distributing system.

The strainers are what are known as the No. 8 Cook well-point strainers and are 16 ft. long, 36 in number, giving a total length of 576 ft. The openings in these strainers were cut with a tool 0.008 in. thick, at right angles to the axes. Mr. Williams stated that probably a somewhat larger opening would have been more suitable for the kind of material in which the strainers are laid.

As at the time of the examination the water in the river was about 10 ft. deep over the bar in which the intake is laid, it was impossible to examine the filtering material, which is said to consist of clean sand and fine gravel and is so located in the stream that there is little, if any, deposit of mud on the surface. The intake is at about the low water mark of the river, and the material over it is thus constantly submerged.

The accompanying diagram shows the layout of the strainers with the cast-iron collecting and suction pipes. It will be seen that the water enters the strainers, then blows through an 8-in. cast-iron pipe into a 16-in. pipe, and thence into the main 20-in. cast-iron suction leading to the pumps. The total length of the strainers, as stated, is 576 ft. and they probably drain an area of approximately 4,500 sq. ft. Assuming the capacity of the sand bed to be 16 gal. per square foot per minute (as determined by previous tests), the maximum quantity of water available by the strainers is 72,000 gal. per minute, or 10,000,000 gal. per day, assuming the entire area equally efficient. At the time of the examination, water was being pumped at the rate of 1,200 gal. per minute, or about 1,500,000 gal. per day, showing that the estimated capacity of the intake provides for a very considerable future increase of consumption. The strainers are patented and were purchased at a total cost of $2,000.

On the diagram it will be noted that 16-in. and 20-in. emergency intakes are provided, which are intended to be used in case something happens to the strainers. As has been noted, there is no storage or distribution reservoir, and in case the strainers become clogged or for any reason do not give sufficient water, it is intended to back flush them by pumping direct, using one of the emergency intakes. The plant has been in use since July 1, 1906, and it has never been found necessary to back flush.

One interesting feature developed in operating the plant is that the suction lift seems to be independent of the stage of water in the river but varies only with the quantity of water pumped. Even with very high stages of the river, the ordinary vacuum of 3 in. is maintained when the pump makes 30 r. p. m.; but when increased to 34 r. p. m. the vacuum is 4 in. This is true also at low stages of the river, and it would seem that there is but little loss of head in the filtering process itself, the actual movement of water into the strainers being very slow, but that the loss of head developed while pumping is due to friction in the suction pipe. This is a very interesting fact, as it would be expected that ultimately the capacity of such systems would be limited by the amount of water that could be drawn through the sand, a quantity which depends finally on the suction lift at the pumps. It was stated that the suction at the pumps had not increased since the beginning of operations, and the fact that it has never been necessary to back flush and clean the sand would show that there is but little decrease in the capacity of the strainers.

After the plant was first started in operation it was found that the filtered water was occasionally turbid, and an inspection showed that proper attention had not been given by the contractor to leveling the sand and gravel after back filling over the strainers, so that there were numerous places where the filtering material was not of sufficient depth to give proper purification. In December, 1906, more filtering material was added and the river bed leveled off, since which time the plant has been in successful operation giving a very satisfactory water up to the present time.

The plant was operating under normal conditions at the time of the examination, the river being fairly turbid and rising, dut to a rain of the previous night. Only four samples were collected: One from the river, one from a tap at the pump, and two from a tap at the force main.

The following table shows the analyses made and the results obtained in parts per million:

| | River Water. | Tap from Pump. | Tap from Force Main 1 | Tap from Force Main 2 |
|---|---|---|---|---|
| Turbidity | 170 | 2 ± | 2 ± | 2 |
| Color | 22 | 15 | 15 | 15 |
| Carbon dioxide | 2 | 14 | 12 | .. |
| Alkalinity | 75 | 95 | 95 | 95 |
| Incrustants | 29 | 18 | 32 | 25 |
| Total hardness | 104 | 113 | 127 | 120 |
| Iron | 1.6 | 0.7 | 0.2 | 0.3 |
| Chlorine | 13.0 | 12.2 | 9.5 | 9.5 |
| Bacteria per c.c. | 26,500 | 29 | 29 | 21 |
| Colon in 1 c.c. | Yes | No | No | No |
| Colon in 100 c.c. | Yes | No | No | No |

The bacterial plates were made in the field, as were also the chemical determinations of turbidity, alkalinity, color, and carbon dioxide. The remaining analyses were made in the laboratory.

The above analyses show a remarkable purification of the river water, the bacteria being practically all removed by the filtration. The slight increase in total hardness and of carbon dioxide indicates a small percentage of ground water, but it will be noted that the filtered water contains even less iron and chlorine than the water in the river. There were a large number of bacteria in the raw water, and also the presence of considerable pollution is indicated by the coli results. No coli, however, were found in the filtered water.

It will be noted that there is not a very great removal of color, the reduction being but seven parts per million.

The samples of filtered water were seen to contain small particles of suspended matter, and a subsequent microscopical examination in the laboratory revealed the fact that there was present a considerable quantity of growth similar to Crenothrix. No trouble has yet been experienced with odors or tastes in the supply, but it is quite probable that in the future considerable trouble may develop due to the luxuriant growth of this plant, causing a subsequent closing of part of the strainers and a diminution of the capacity of the intake. Owing to the lack of storage of filtered water, it is necessary to run the pumps continuously, and it will be very difficult to treat the intake for the removal of any such growth.

In conclusion it should be said that the present intake of the Owensboro Water Co.'s plant has for about one year been furnishing a clear, satisfactory water obtained by the filtration of water from the river with no greater expense of maintenance than would be incurred by the pumping of untreated river water. At the time of the examination the supply was found to have a remarkably low bacterial content and to be in every way satisfactory as a municipal supply. The probability of future trouble by the clogging of the strainer system by growths very difficult of removal is apparent, a condition which might render the present supply very unsatisfactory.

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A CORRECTION should be made in one of the values of the coefficient of discharge in the table on page 354 of this issue, being part of the article on "Experiments with Submerged Tubes 4 Feet Square." Under a head, *h*, of 0.20 ft. and form of entrance and outlet of type *e*, the coefficient for length of tube of 2.50 ft. should be .694 instead of .684 as printed.

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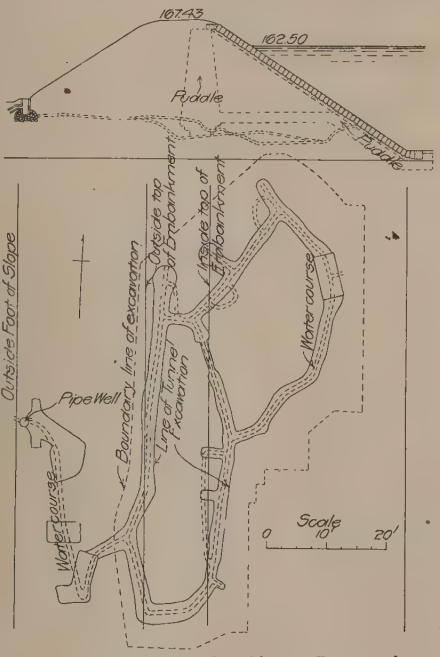
## Repairing a Leak at Hope Reservoir, Providence.

Mention was made last year of the fact that a leak which has existed for over 25 years in the embankment of Hope reservoir at Providence, R. I., had been giving the city engineering department quite a puzzle before its course was finally traced. This reservoir is a service basin surrounded by dwellings. The leak appeared originally at the foot of one of the slopes, where a pipe well was constructed to intercept the seepage and direct it into a street sewer. Measurements of the discharge were kept for many years, and during the last few years the amount had increased somewhat, although not enough to cause alarm. Examinations had been made a number of times and considerable time and money had been expended, without finding any clew as to the location of the point on the inside of the bank where the water escaped. In May of last year the reservoir was drawn off for the purpose of cleaning it, and advantage was taken of this opportunity to investigate and repair the leak. The work is described in the 1906 report of Mr. Otis F. Clapp, city engineer, from which the following notes are taken:

It was thought that the best way to trace the course of the water would be to follow the stream back into the bank. When the water had been drawn down so that there was a head of about 1 ft. on the discharge, digging was commenced. Almost immediately a water channel was found, measuring from 2 to 4 in. high and from 4 to 8 in. in width, and nearly level. This was followed some 54 ft. in open trench and then tunnelling was commenced. When a point was reached about under the outer top edge of the embankment the channel divided and ran in nearly opposite directions. In the belief that time could be saved by so doing, a lot of colored water was forced back into the reservoir by way of each of these two routes, but the color appeared at the same point on the surface of the water in the reservoir.



Seepage Channels, Hope Reservoir.

The reservoir was then emptied and cleaned and the tunneling was continued by way of both branches. The stone slope wall was taken down where the colored water had appeared and a hole was found several inches in diameter, which went back into the bank for several feet, where it branched and ran in opposite directions. The tunneling was prosecuted until the lines met, every effort being made to discover any side openings that might lead away, but only a few were found and those followed to the end. The material of the embankment was found to be hard and in the best of condition, only a few soft places were found, and those, all near the water course, were completely removed. The tunnel averaged about 4 ft. high and 2½ ft. wide, just large enough for a man to work in. This tunnel was surveyed and platted, and a sketch of it is reproduced here. The tunnel measured 251 lin. ft., with 54 ft. of open cut. There were discovered 359 lin. ft. of water course in embankment.

It was not thought safe to refill the tunnel, and so that portion of the bank lying over the tunnel excavation was taken out and the embankment rebuilt. The water channel was found to have been about on the surface of the original soil, or where the artificial work in building the embankment was begun. In repairing, the precaution was taken to go down lower in the line of the core wall, finding rock from 2 to 5 ft. below where the embankment had been started before, and filling in with cement concrete. The excavation was then refilled with great care, some three carloads of clay being added to the soil taken out before replacing.

The work of repairing took about 3½ months to perform, and no sign of any leak has since appeared at this point. Mr. Samuel M. Gray, who was resident engineer at the time the reservoir was built, was consulted in relation to the repairs. A smaller leak or spring, co-existent with the other leak, is still discharging near the corner of the reservoir and will be examined later.

## Strengthening an Old Lattice Girder Bridge.

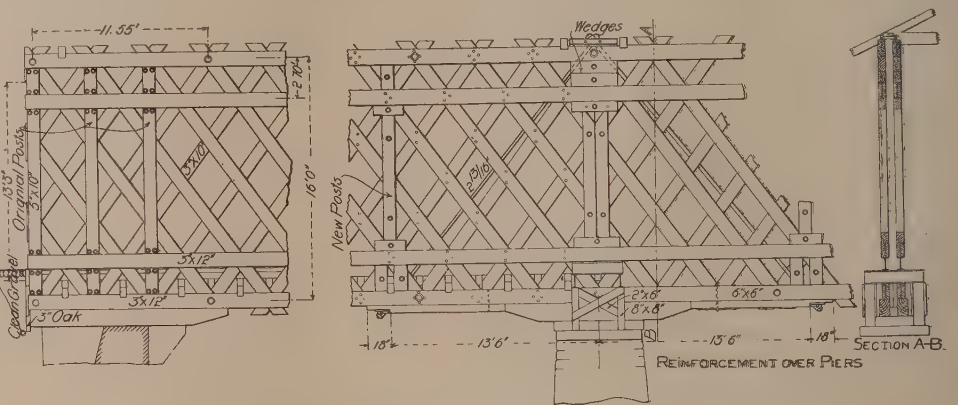
By George Jacob Davis, Jr.

The city of Grand Rapids, Mich., is divided into two parts by the Grand River, which is spanned by a number of bridges, two of which were originally timber lattice girders. The lattice girder bridge at Bridge St. was replaced by a steel pony-truss bridge to accommodate street cars and these trusses were, in 1904, replaced by a reinforced concrete arch bridge. The lattice bridge at Leonard St., built in 1879, still stands and carries a very heavy traffic. A few notes on the method used by the designer in computing the stresses in the bridge and on its failure and strengthening may be of interest.

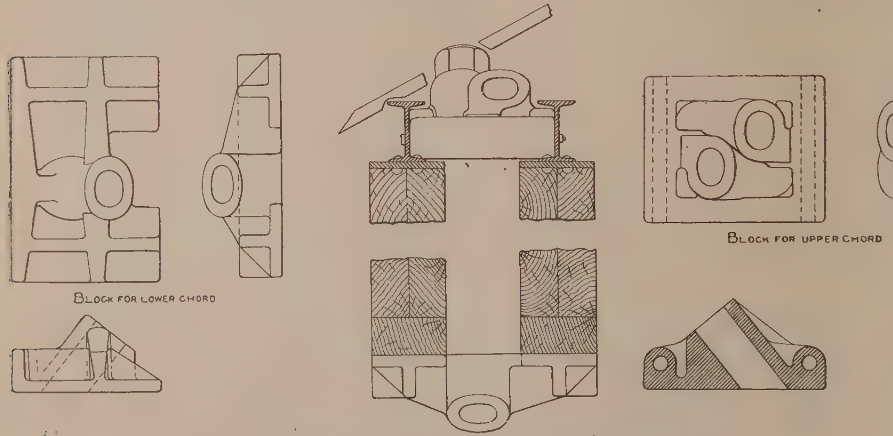
The bridge consists of eight spans of 104 ft. each, built of white pine. The trusses, as shown in the illustration, are sextuple system Warren trusses, fastened together at each intersection with treenails. There were, originally, no vertical posts except three in each truss at each end of the bridge, the trusses being continuous over the intermediate piers.

From the original computations it appears that the dead load was taken as 1,000 lb., and the live load as 1,848 lb. per running foot of bridge, on two trusses. Taking the distance between centers of gravity of chords as 13.3 ft. and the span as 104—5 = 99 ft. (piers are about 5 ft. wide on top), and considering the girders as simple beams, gave the computer a stress of about 131,300 lb. on each chord, of which 78,800 lb. would be carried by the outer boom and 52,500 lb. by the inner boom. Calling the girders continuous would reduce the maximum strain to about 40,000 lb. or 800 lb. per square inch, only 48 sq. in. being considered available at joints in the booms. In taking care of the shear the computer assumed all the diagonal strains to be carried by one system and delivered to the abutment by one strut giving a load of about 70,488 lb. per truss on the abutment or 70,488 sec. θ = 93,987 lb. as the approximate strain on the diagonal, which was too great, but by dividing by six (the number of diagonals cut by a vertical section) gave roughly 15,500 lb. to be carried by one diagonal.

Assuming the 3x10-in. white pine diagonals to have an ultimate strength of 95,190 lb. and considering them to act as long columns, 3 ft. long, they were found to have a factor of safety of about six when the truss was considered as a simple beam. This factor of safety is still further increased by virtue of the fact of the truss acting as a continuous girder, but it is diminished by virtue of the fact that the diagonals are not rigidly supported at 3 ft. intervals, it being possible for several of them to tend to bend in the same direction, thus increasing the effective length of the columns. This action has, in fact, taken place. In some cases several of the diagonals have bowed, perhaps 1½ in., while in others, they are bent into an S form.



End of Lattice Girder Bridge and Truss at Pier.



Blocks Used at Ends of Reinforcing Rods.

The chords each consist of two booms of four 3x12-in. timbers, each 30.8 ft. long. To determine the length required it was assumed that each timber would carry a stress of 27,000 lb., giving 1,500 lb. per square inch on the net area of 18 sq. in. which remain after deducting for the three 2-in. pins which were intended to be used. It will be noted above that the designer decided that only about 40,000 lb. would come on the two halves of the boom or 20,000 lb. to be transmitted past a joint by a single timber. Figuring the strength of the pins in single shear as 500 lb. per square inch, or 1,500 lb. per pin, gave a safe stress of 4,500 lb. per set of three pins. Hence to carry the 27,000 lb. past a joint six sets of pins were required. The sets are 11.55 ÷ 3 = 3.85 ft. apart. This would require a plank 2x6x3.85, or 46.2 ft. long, plus something for end shearing. To reduce the length, clusters of four 1¾-in. pins were put in instead of three 2 in., the area of pins also being increased thereby from 9.42 to 9.6 sq. in. These pins also connect the lowest boom to the diagonals and they are here considered to be acting in double shear.

No clue is given in the old notes as to how the number and size of pins required to connect the diagonals to one another was determined.

A roof which covers the bridge has perfectly preserved the timbers except the ends of the floorbeams, which extend out to support the sidewalks and are thereby subjected to periodical wettings by rain.

The trusses have proved to be amply strong to resist the bending stresses. In the middle of the spans where the bending moment is greatest, there is no distortion noticeable in either diagonals or booms, the pins apparently have not sheared and the original camber still exists. Over the piers, however, the trusses have failed by shearing, allowing the bridge as a whole to settle approximately 0.2 ft. The floor, being supported by the lowest boom, which could not settle over the piers, shows ugly humps at these points. The failure occurred partly by the diagonals buckling between chords, as indicated above, and partly by the crushing and twisting of the lowest boom and probably a shearing of the pins connecting it to the diagonals or a distortion of the lower end of the diagonals. Of the 0.2 ft. by which the upper chord has approached the pier only about 0.02 ft. on an average was produced by the buckling of the diagonals, the remaining 0.18 ft. being the result of the other failures. The 8x14-in. bolsters and lowest boom are bent into an arc. The upper chord seems to retain very closely its original shape and the distortion in the lower chord extends only about 10 ft. each side of the piers.

In reinforcing the bridge no attempt was made to bring the trusses back into shape, but only to relieve them of some of the stress and to prevent any further failure at these points. Jacks were put under the upper chord and screwed up, thereby stretching the chords apart slightly, and columns were then slipped in between the chords with very tight fitting joints. I-beams were next inserted under the upper booms of the lower chord and wedged up tight. Then the 2 13/16-in. round bars were placed and tightened until the new parts took a firm bearing and the bridge showed a slight rise on each side of the pier. The humps in the floor were removed by dressing down the stringers when the new flooring was laid.

Rankine states that lattice-work girders were first introduced by Mr. Ithiel Towne. They are not economical of material, being of the same strength in all parts regardless of the variation of the stresses. They were, however, easily and cheaply built, but have fallen into disuse on account of the growing scarcity of timber. Lattice trusses of iron and steel are still used somewhat in Europe. From the experience with this bridge it would seem that the introduction of posts over the piers when the bridge was built, in that way distributing the stresses from the pier reactions more uniformly among the various systems of diagonals, would have prevented the failure. The bridge is now apparently good for many more years of service.

The bridge was designed and built by Mr. Wm. Seekel, who was then city engineer of Grand Rapids. The recent reinforcement was done under the direction of the present city engineer, Mr. L. W. Anderson, the author being in charge of the work.

**The Large Traveler** used in erecting the Blackwell's Island bridge across the East River at New York, was injured by lightning on Sept. 23.

## The Erection of the South Cantilever Arm of the Quebec Bridge.

The design, fabrication and erection of the Quebec bridge have been fully explained in previous issues of this journal, and those who wish to become familiar with the structure are referred to the list of articles which closes with this description of some details of the erection of the south cantilever arm, not previously published.

The erection of the cantilever arm was commenced with the forward bent of the traveler in the plane of the main vertical posts. The forward 69½-ft. cantilever trusses were secured to the foot of the traveler and their extremities were connected by the transverse girder 81 ft. long, which weighed 25,000 lb., and was handled from the overhang of the traveler by two pairs of wire rope tackles converging to a swivelled attachment at the center point. The top of the traveler was temporarily braced to the top of the permanent superstructure in order to avoid special anchorage at this point and the first panel of the cantilever arm was erected in 37 operations made with the traveler in the same position, which was the fifteenth counting from the commencement of the erection of the south anchor arm.

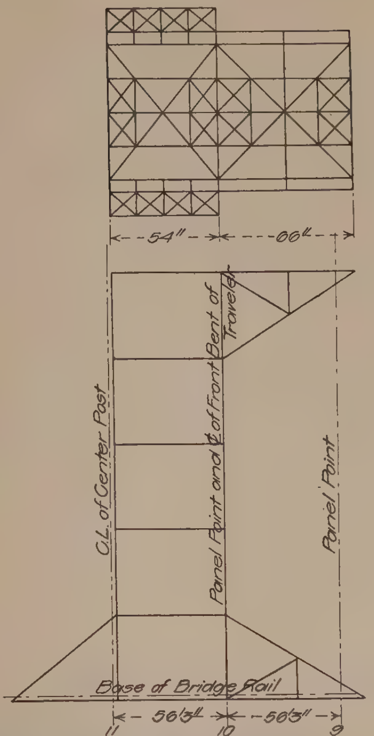
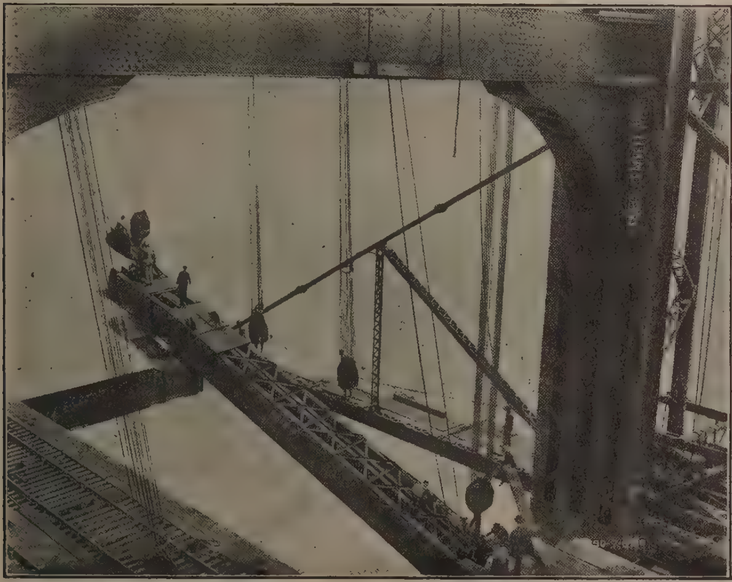
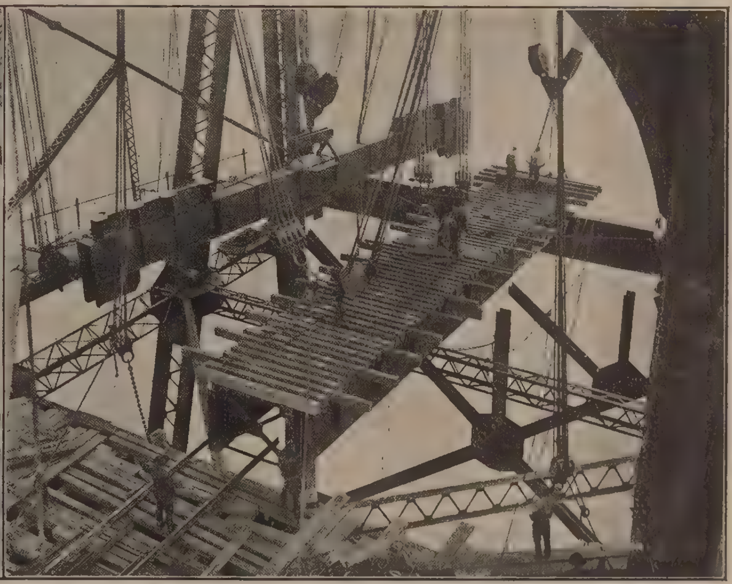


Diagram of Traveler.

The erection superintendent was furnished with a blue-print showing the sequence and detail of all these operations and illustrated with dimension sketches giving the lengths and widths of all their principal members and showing where and how the tackles were attached to them. A number of diagrams were also given to show the positions of the different tackles suspended from the traveler and their capacities and the reference letters by which they are designated in the description, R and L, always meaning right and left hand.



Erecting First Panel of Cantilever Lower Chord.



Handling Track in Cantilever Platform of Traveler.

Preparatory to the first operation a pair of stringers and its track, making a unit 69½ ft. long weighing 60,000 lb., was delivered on cars in the tower of the traveler and hoisted by one 22-ton tackle EC and two 17-ton tackles MC, both attached to it at the center of gravity. Tackle EC was slacked off to fleet the girders forward under tackles MC which fleeted it transversely into position. The duplicate pair of track stringers on the opposite side of the bridge axis were similarly put in place and the first panel, No. 10, of bottom chord for each truss was delivered on 2 cars. Each piece was 70 ft. 7 in. long, weighed 195,000 lb. and was supported, 19 ft. from the forward end, on a bolster in the middle of a flat car 40 ft. 2 in. long over all. The rear end of the chord was supported on a bolster just in the rear of the forward truck of the rear car and was held back by a horizontal preventer tackle made with a 1½-in. rope rove through 14-in. blocks to exert a maximum pull of 15,000 lb. Pivoted attachments described in a previous article were secured to both ends of the chord projecting above its top flanges to receive two wire rope tackles HR and HL at the rear and two wire ropes tackles AR and AL in front by which it was unloaded, lowered and fleeted to position as shown in the photograph. After the lower end was splice bolted to the special V-shaped section engaging the shoe and pedestal, tackles H were released and the upper end was sustained by tackles A.

The first section LSB5, of each diagonal, R and L 63 ft. 9 in. long and weighing 29,000 lb., was unloaded by one tackle HR or HL attached to the center of gravity just above the center of gravity by a double 1-in. chain, snubbed to position with a double 1½-in. manila line and bolted securely at the lower end.

The lower horizontal struts SS6, R and L were each swung to position with two 1½-in. single foot lines and the connections at both ends bolted, thus completing the triangle and holding the diagonal members stable.

Temporary suspension bars were attached to the outer ends of the lower chord sections 10 R and L, releasing the tackles A, which were detached after the positions of the chords were adjusted.

The four bottom lateral members, not shown in the diagram, were erected with 1½-in. lines rove through single and double blocks and attached to their centers of gravity with ⅞-in chains.

In the sixth operation the second section MSP5 of the diagonal, 56 ft. long and weighing 22,000 lb., was unloaded and swung to place with a pair of 17-ton tackles, MR and ML attached a little above the center of gravity by a double 1-in. chain. The lower end was securely splice bolted and the tackles released.

The horizontal struts SS4, R. and L., were swung to position with single 1½-in. whip lines and securely bolted.

Transverse struts H2 and laterals N3 in the plane of struts SS4, not here shown, were swung to place with single whip lines attached at the center of gravity.

The third sections USP5, R. and L. of the diagonal, 80 ft. long and weighing 22,000 lb., were swung to position with 1 wire rope tackle FR or FL and a double 1½-in. snub line attached to double 1-in. chain a little above the center of gravity of the member. The bottom end was spliced bolted and the top end sustained in position by the tackles.

Struts SS8 R. and L. were swung to position with single whip lines and bolted at both ends.

The upper sections UT5 R. and L. of the main diagonal, 81 ft. 9 in. long and weighing 92,000 lb., each were swung to position by two pairs of wire rope tackles HR and L attached to the upper ends with a pivoted yoke and to 17-ton tackles M and MC attached to the lower end with a 1-in. chain.

Struts SS7 R. and L. and the transverse strut H3 and lateral 11 in the same horizontal plane were swung into position with single whip lines attached at their centers of gravity with 1-in. chains and bolted in position at both ends.

The lower sections LT6 R and L of the main diagonal, 81 ft. long and weighing 64,000 lb. each, were hoisted with two pairs of wire rope tackles, FR and FL, with special attachment to the member a little above the center of gravity, and temporarily held suspended by the tackles.

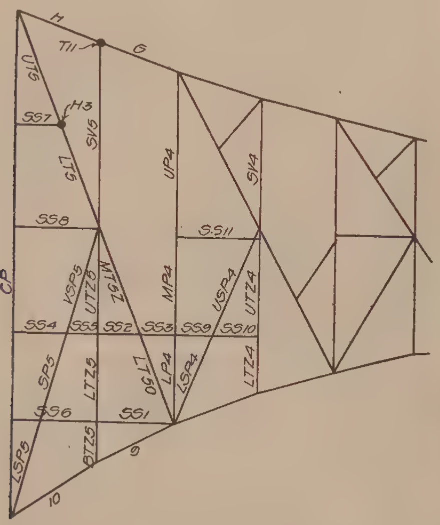
The upper ends UT5Z R and L of the hanger T5Z, 100 ft. long and weighing 55,000 lb. each, were hoisted by two 30-ton tackles EC at one end, attached to the lower end by a 1-in. chain and by two 55-ton tackles AR and AL with a special bent plate attachment at the upper end.

The transverse strut T1, not here shown, and the longitudinal strut SS5 R and L were hoisted with single whip lines attached with ¾-in. chains.

The middle sections MT5Z R and L of the hangers, 54 ft. 9 in. long and weighing 21,000 lb. each, were swung by one pair of manila rope tackles X at one end and another pair of wire rope tackles HR and BL at the opposite end are all attached to it by 1-in. chains.

The hangers LT5Z R and L at panel point 10 were swung with single tackles attached with a 1-in. chain. All of the horizontal longitudinal struts SS, etc., were connected with temporary bolted plates permitting them to be adjusted as the panels of the truss varied during erection. The transverse struts NN, etc., were also assembled with temporarily bolted connections at one end until the main trusses came to their final adjustment.

The temporary suspension bars were removed from the river ends of the sections 10 R and L of the lower chords and the working platform on the left side of track on the cantilever of the traveler was removed. The first false-work transverse girder, 106 ft.. long and weighing 108 tons, was fitted with bearing girders, chord blocks and eye-bar suspenders and swung with two wire rope tackles, HR and HL, AR and AL connected to it with a special swivelling attachment web-connected to a shoe-like device bolted to the top flanges of the girder. It was hoisted by operating the tackles HR and HL after which it was swung to one side and HL was cut loose. It was then pulled forward with tackles FR and AR and tackles HR and FR were cut loose after which tackles BL was connected to it and it was lowered to position.



Location Diagram, Cantilever Arm.

Struts T6, T3, T4, T5 and T12 and diagonals T1, Too, T1 and To, all members of trussed floor beam 10, were each swung to position with two sets of manila rope tackles.

The top chord of the trussed floor beam at panel point 10, 65 ft. long and weighing 32,000 lb., was swung by a pair of 17-ton tackles MC and wire rope tackles BL and BR with a swivel connection to a horizontal beam having vertical bent plate hooks engaging the top flange of the floor beam at two points equally distant from the center. It was unloaded from the car by operating the tackle MC, drifted forward by operating tackles MDR and DL and lowered to position after the removal of the temporary track.

The two pairs of the stringers in the traveler cantilever were removed by two tackles MC and one tackle EC and were fleeted longitudinally with tackles EC and placed on cars.

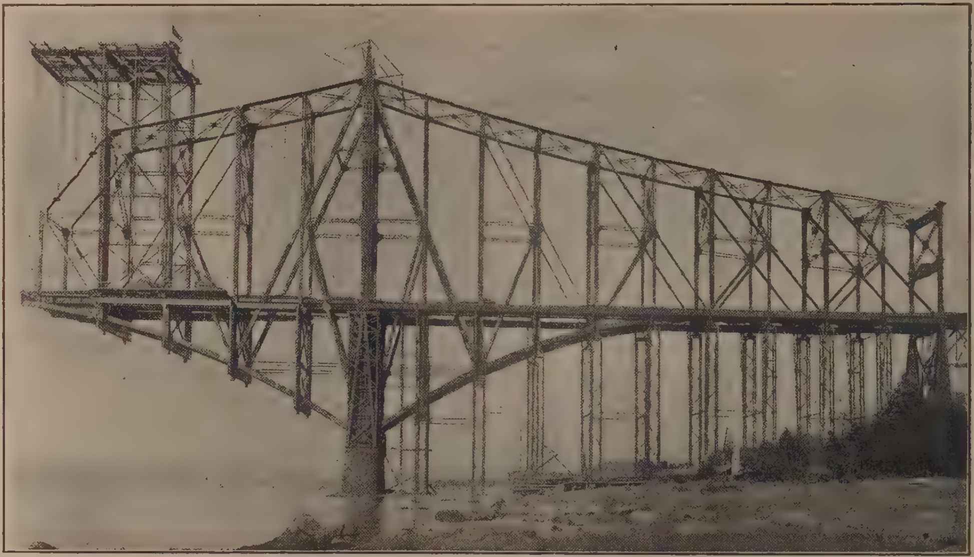
Bottom laterals, diagonals N4, were lowered to position with single whip lines.

The permanent stringers were braced together in pairs at the storage yard, and one pair of railroad stringers was swung to position with two manila rope tackle EC and MC. Track was immediately laid on these stringers and cars were run out on them bringing the other stringers which were swung to position by manila tackles MC and M attached at the center of gravity.

The sub-verticals, SV5 R and L, 103 ft. 5 in. long, and each weighing 17,000 lb., were swung to position with a manila rope tackle at the lower end and two wire rope tackles WR and VL at the upper end both attached with 1-in. chains.

Lower diagonals T7, T9, T10, struts T2 and upper diagonals T8 and T9X and T10X, forming transverse bracing below sub-verticals SV5 R and L, were swung to position with single manila rope tackles.

In panels 10 of the anchor arm and cantilever arm the top chord bars were handled in three separate groups of eight each, as the space between the tops of traveler and the top chord was not quite sufficient to use the special clamps designed for the handling of the whole groups nor to fleet a group of bars over 10 ft. wide enough to clear the other twin members. In all other cases the bars were assembled in the yard and hoisted into place in one body.



Traveler Erecting Cantilever Arm from Suspended Falsework.

The twenty-four 16-in. eye-bars in the top chord of panel H were accurately spaced and clamped together in three groups of eight bars each at the storage yard. The first outside group was swung from the cars with blocks X, MC and V and delivered to two 6-part wire rope tackle suspended from the traveler at each end of the panel with one tackle M on the overhang. The group weighing about 43,000 lb. was unloaded from the car by tackles X, MC and V suspended from the center line of the bridge and fleeted transversely into the planes of the trusses by two 6-part tackles and one 17-ton manila tackle M on each of the outside jigger beams of the traveler overhang. Tackles X, MC and V were then released and the bars were hoisted and fleeted longitudinally to position by corresponding tackles on the outside jigger beams and by the 6-part tackles at the ends of the eye-bars. The 6-part tackle at the upper end of the eye-bars was released and the bar was temporarily connected by a round timber through its pin hole. The center group of bars was swung from the cars with tackles FR and FL at the lower end and tackles X and MC at the upper end and were hoisted and fleeted to position with tackles M and a 6-part tackle suspended from the outside jigger beam. It was then drifted longitudinally to position with a 6-part tackle suspended from the outside jigger beam and a temporary pin driven as far as possible through the four holes. The third group of I-bars was swung from the cars and hoisted to position in the same way as the first group and the permanent upper pins fully driven and the temporary pin driven through the lower hole. In erection all of the bars were drifted longitudinally toward the river sufficiently to clear the main post with their upper ends and the lower ends were lifted temporarily above the top of sub-vertical post SV5. The upper ends were then drifted back longitudinally between the webs of the main post cap and the lower ends lowered to position for connection with the top of post SV5.

Top lateral diagonals L12, L13 and L14, not shown in the accompanying diagram, were swung to position with single manila tackles and the tracks and wheel stops were placed on the other stringers of the cantilever platform.

The inside vertical falsework post, 79 ft. 8 in. long and weighing 17,000 lb. each, of the first transverse bent of suspended false-work was handled by one 8-ton manila tackle X, attached with a ¾-in. chain at the lower end and by one 10-ton manila tackle U and one 9-ton manila tackle S attached at the upper end with a 1-in. chain and after placing it into position bolted fast to the suspended transverse beam and to the truss.

The outside vertical posts of the same bent weighing 9,000 lb. each were handled by the same tackle and pulled out into position by a single tag line attached to them and rove through a snatch block on the end of an outrigger timber.

The right and left inside traveler stringers, weighing 17,000 lb. each, were swung into position with tackle M attached at the center of gravity by a 1-in. chain and fleeted into position by tackle S at the forward end and an outhaul tackle at the rear end.

The next operation was to move the traveler one panel forward until its front posts were at panel point 10 in the sixteenth position, after which the erection of the next panel of the cantilever arm was commenced with the two right and left lower chord sections, 9R and L. The number of operations for each panel was reduced to correspond with the reduction in the number of pieces. All main truss members not exceeding 140,000 lb. were erected simultaneously for both trusses, with the exception of a few cases in which the traveler was holding up other truss members when handling these pieces.

The traveler was equipped with about 60 tackles including whip lines, all of which were operated by the eight drums and the 16 capstan heads of the four electric hoisting engines. The heaviest loads hoisted at one time were the pair of 100-ton bottom chord sections and the pair of 70-ton top chord sections simultaneously raised. All of the principal tackles were designated by numbers and signals for their operation were transmitted by the foreman to the enginemen by various gestures of the arms and head repeated a corresponding number of times. This system was so well perfected that the hoisting was done with ease and regularity and with remarkable silence, the engines running so smoothly under heavy loads that they could scarcely be heard. The work was also much facilitated by a complete telephone system connecting the superintendent's office, the traveler, the storage yard, the city of Quebec and the bridge shops at Phœnixville.

The lower cantilever working platform weighed complete about 80 tons and as already described the two pairs of stringers with their tracks were handled as separate units, each hoisted by four 8-part tackles which could remove them and place them on cars in the traveler tower in about four minutes. They were then run back to the storage yard while the permanent bridge floor was erected, after which they were returned and replacd in the traveler in its next position as already described. After the traveler had advanced to the fifth panel of the suspended traveler false-work the four vertical posts, the longitudinal girders and tracks on both sides, weighing about 40,000 lb. each, were hoisted with four 8-part tackles each in four minutes, and delivered to cars which carried them back to the storage yard until they were required for service in the next panel in advance.

The four vertical posts of the rear transverse bents were similarly hoisted, loaded on cars on the permanent bridge floor, and also sent back to the storage yard where they kept until required for use on the opposite side of the river. The suspended transverse girder was then disconnected from the lower chord pin and lowered by four 9-part tackles to a 28x100-ft. scow anchored directly below. The scow was then moved forward to a position under the fifth panel point and anchored there while the transverse beam was hoisted up and connected to the lower chord pins. Its elevation here was so near that of the track level that the vertical columns in the transverse bent were replaced by short steel blocking to receive the forward ends of the traveler track girders which were then brought back again from the storage yard and swung to position by tackles suspended from the traveler. During erection the utmost security was afforded by invariably filling every field rivet hole with a service bolt until the rivet itself was driven. Both trusses and traveler developed great rigidity and and the latter was so stable that when exposed to a wind of 55 miles per hour the upper part had an observed deflection of only 13 in.

Steel was erected in this cantilever arm with greater rapidity than it had ever before attained on cantilever erections. On Aug. 1, 1906, 410,000 lb. of steel were erected on the south cantilever arm and the maximum day's record was 340 tons when four bottom chord sections were erected for the south anchor arm. The significance of these amounts is emphasized by their comparison with the weight of an ordinary 200-ft. double track railroad bridge span which is less than the amount erected in one day on the cantilever arm with the pieces assembled in midair nearly 200 ft. above the water level.

As a matter of convenience to those who wish to study this structure, the following list of articles concerning it published in this journal is here given:

Anchorage and main piers, July 27, 1901.

Approach to anchor arm (still standing), April 11, 1903.

General methods of erection, March 4, 1905. This article illustrates the steel falsework for erecting the anchor arms and the various travelers.

The general design of the bridge—Part I, April 1; Part II, April 8, 1905. These articles outline the methods of calculation, describe the general features of the design, illustrate the anchorage and the portal bracing, and explain the chief features of the main and intermediate vertical posts, shoe and pedestal, bottom chords and eye-bars.

Progress on erection of the bridge, Sept. 16, 1905. Illustrates some of the important members.

Progress on the erection of the bridge, June 23, 1906. This article contains some important illustrations giving an idea of the eye-bars and their assembly.

Steel falsework used in the erection, Sept. 8, 1906.

Camber adjustments made in the erection, Sept. 15, 1906.

Handling members during their erection, Sept. 22, 1906. Illustrates some of the special devices for this work.

The anchor arm, Dec. 1, 1906. Gives strain sheet and elevations of various bents, and notes on methods of design.

Notes on the fabrication of the bridge, Dec. 15, 1906. Describes shop methods.

Adjustable connection of anchor arm lateral system, Dec. 29, 1906. Illustrates one of the most interesting details.

Anchor pier towers, Jan. 12, 1907.

Erection attachments for bottom chords and vertical posts, Jan. 19, 1907.

Erection of the main vertical posts, Jan. 26, 1907. Describes some of the special equipment for the purpose.

Handling eye-bars, Feb. 9, 1907. Gives sketches of important eye-bar connections.

Main traveler for the erecton, Feb. 23, 1907.

Erection of the south anchor arm, March 2, 1907. Illustrates some of the special equipment employed.

Vertical bent on main piers, June 22, 1907.

Pedestal and shoe on main piers, July 6, 1907.

Upper part of center posts, July 13, 1907.

Upper part of typical intermediate post, July 20, 1907.

Details of intermediate posts, July 27, 1907.

Details of diagonal members, Aug. 3, 1907.

Bottom chord details, Aug. 10, 1907. This article gives illustrations of members which have received adverse criticism in letters to this journal.

Floor system, Aug. 17, 1907.

Transverse bracing, Aug. 24, 1907.

The collapse of the bridge, Sept. 7, 1907, Current News Supplement.

Illustrations of the wrecked bridge, Sept. 14, 1907, Current News Supplement.

Suspension falsework of the cantilever arms, Sept. 21, 1907.

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The Electric Train Operation contemplated on the Pacific Coast extension of the Chicago, Milwaukee & St. Paul Ry. will probably start near the east end of the 8,000-ft. tunnel through the Bitter Root Mountains and extend to some point west of Tekoa, Wash. Power will be furnished by developing sites along the St. Joseph River between Ferrel and North Fork, where something like 180,000 h.p. can be obtained if desired. It is understood that this power may be developed by an independent company, as in the case of the Western Pacific work, and the surplus sold at Spokane and in the Coeur d'Alene district.

## The Melwood Avenue Retaining Wall, Pittsburg.

The Department of Public Works at Pittsburg, Pa., has now in course of construction on Melwood Ave., a reinforced concrete retaining wall about 220 ft. long which has a maximum height at the center of 64 ft., exclusive of the 4-ft. parapet wall. The wall spans a gulley which was formerly crossed by a timber trestle bridge. Since the street is an important thoroughfare at that end of the city, when the old timber structure decayed the problem was at once taken up of replacing it with a heavier and more permanent structure. Bids were called for on a steel viaduct and on a reinforced concrete retaining wall and resulted in the selection of the latter. The avenue runs approximately east and west and the front face of the wall forms the northern line of the street, the gulley back of it being filled in.

Briefly stated the wall consists of a reinforced concrete face and footings or floor, with reinforced counterforts at frequent intervals. The character of the grounds introduced some complications into the design of the footings and between successive counterforts hardly any two slopes of the footings are alike, the maximum being about 38 deg. parallel with the length of the wall, and about 30 deg. transversely. The footings were sloped transversely instead of being made horizontal not only to save excavation but also to save concrete. The ground consists of a layer of loose material beneath which is a bed of excellent shale on which the footings are founded.

The floor between the counterforts is 2 ft. thick, the front wall 18 in. thick through its full height and the counterforts 1 ft. thick. The concrete in the floor and the foundations consists of 1:2½:5 mixture and in the face wall, parapet and counterforts a 1:2:4 mixture, the sand being washed river sand, and the aggregate, gravel. The reinforced concrete portion of the wall has not been carried across the entire length of the hollow, there being a plain concrete gravity section 18 ft. long on the east end and 22 ft. long on the west end. Two vertical expansion joints have been provided at approximately the third points in the length of the wall, 74 ft. apart. These expansion joints are V-shaped and filled with three thicknesses of heavily-tarred paper.

The reinforcement consists of round rods varying in diameter from ½ to 1⅝ in., and anchored to plates imbedded in the floor and in the face of the wall by nuts and pins. These anchor plates are all ⅝ in. thick and are 8, 8½ or 11½ in. wide depending upon the height of the wall where they are used. One of these plates runs the entire length of the floor, imbedded in the latter under each counterfort and another plate is imbedded vertically in the wall in the same plane with the floor plate and the center line of the counterfort. These two plates are tied together by round rods varying in diameter from 1¼ in. at the lowest section of the wall to 1⅝ in. at the highest, their spacing and number being naturally dependent on the earth filling which they are to retain, the weight of the latter varying, of course, with the height of the section. At each end of these rods there is a forked eye and the connection is made to the plates by means of pins which after being driven are held in position by two split cotter pins. In the lowest section of the wall twelve 1-in. round rods are used, their spacing in the anchor plate imbedded in the floor being 9¼ in. on centers. From the floor anchor plates the rods spread out, the pins which connect them to the vertical anchor plates in the wall being 13 in. apart at the bottom and 41½ in. apart at the top, varying gradually between these two values. In the highest section of the wall there are forty-two 1⅝-in. round rods imbedded in the counterfort, their connections to the floor anchor plate being 8¼ in. apart and in the wall anchor plate 10 in. at the bottom and 6 ft. 3 in. at the top, varying, as described for the lower section, between these two values. The pins used in connecting the rods to the plates vary from 1½ to 2⅛ in. in diameter.

The floor rods in any given sections between two adjacent counterforts are all of the same diameter, varying from nineteen 1¼-in. rods where the wall is lowest to fifty-three 1¼-in. rods where it is highest. Rods of smaller size have been used in some of the other panels and the required total section made up by using a larger number of rods. The anchor plates have three lines of holes punched in them, the upper line in the floor anchor plate being for the connection with the tie rods running through the counterfort and the other two lines for the reinforcing rods in the floors. All of the floor rods on one side of the counterfort are connected through one line of holes, the rods on the other side being connected through the other line of holes. Nuts are used on the ends of all of the reinforcing rods in the floor and in the wall. The length of the footings varies from 9 ft. to 29 ft. 6 in. in length, and the spacing of the reinforcing rods in them is always uniform in any given panel. All of the rods are bent downward, and though the tops of the floor anchor plates are about on a level with the floor itself the rods come within a few inches of the bottom of the concrete footings.

The reinforcing rods in the face of the wall are of the same general type as those used in the floor and are passed through the vertical anchor plates, in the planes of the counterforts, and held in place by means of nuts. Each rod spans between two adjacent counterforts. They vary in diameter from ⅞ in. at the bottom to ½ in. at the top, the spacing increasing from the bottom upward to suit the varying pressure of the earth backing. This spacing varies from 3 in. at the bottom of the highest section of the wall to 6 in. at the top of the same section. The number of reinforcing rods in the face varies from 53 in the end panels to 151 at the center.

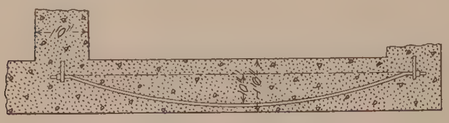
The counterforts are spaced 10 ft. apart on centers except at the two expansion joints where they are on 4 ft. centers. They are all 1 ft. wide and enclose the tie rods which connect the anchor plates in the floor and the face.

The drainage of the wall was very important because of the large run-off from the hollow across which the wall has been built. To provide for the rapid removal of the rain and seepage water, holes have been left through the counterforts at the floor level next to the face, the total area being about 24 sq. in. Since the greatest amount of the seepage would naturally collect at the lowest point or the hollow a 15-in. tile pipe has been laid on the floor alongside of the center counterfort with Y's projecting upward every 2 ft., this pipe extending the length of the footing at this point. The water collecting between the other counterforts drains down through the openings referred to above and discharges into this 15-in. tile which has been connected below the wall to one of the city sewers. The holes through the counterforts have been made by merely inserting cores of wood which were knocked out after the concrete had set. When the wall is completed, the drainage will be further cared for by putting loose crushed stone gravel or cinders on the footings and against the back face of the wall. Two feet of this material will be placed upon the floor, 3 ft. against the back of the wall and 4 ft. over the drainage openings of the 15-in. tile pipe. When this has been placed on the footing and over the tile pipe the earth excavated in order to reach the solid shale foundation will be thrown over it and against the back of the wall, the entire hollow being filled up to the former street level.

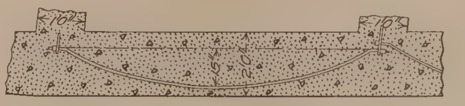
The forms for the concrete work were made of 2-in. lumber dressed on one side and put together in sections. The concrete, in which Phœnix Portland cement was used, was placed in about 3-ft. courses on account of the thinness of the wall and the counterforts and the interference of the reinforcing rods. The front face of the wall was spaded and on its completion will be washed with a neat cement grout.

The materials for the concrete are all delivered at the west end of the wall and mixed on a platform at the edge of the gulley. Long-boom derricks of a combination stiff-leg and guyed type are placed in the bottom of the hollow and pick up the buckets in which the concrete is placed and handle them directly to the points where needed. These same derricks were used in making the excavation and placed the material to one side so that it can easily be picked up and used as back filling when the wall is completed.

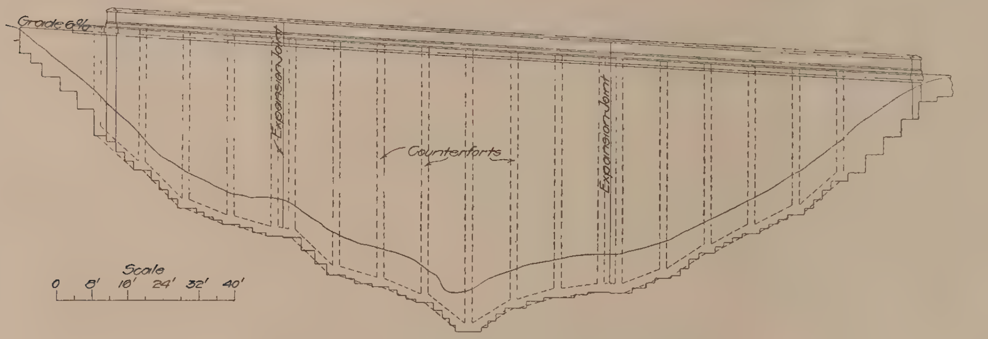
The work is being done by the Bureau of Construction of the Department of Public Works of Pittsburg, of which Mr. Alexander B. Shepherd is Acting Director. Mr. N. S. Sprague, Assoc. M. Am. Soc. C. E., is Superintendent of the Bureau. The wall was designed by Mr. Willis Whited, Assoc. M. Am. Soc. C. E., and is being constructed under his supervision. The contractor is the Penn Bridge Co.



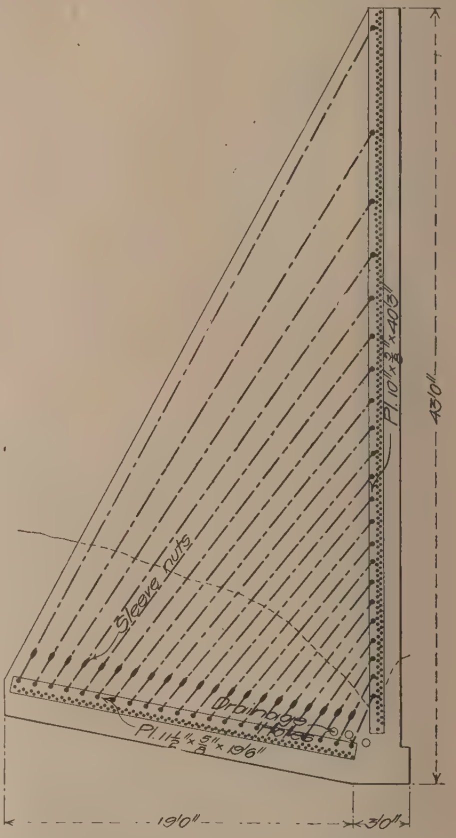
Horizontal Section of Wall.



Section of Inclined Floor.



Elevation of Melwood Avenue Retaining Wall.



Reinforcement of Counterfort, Melwood Ave.

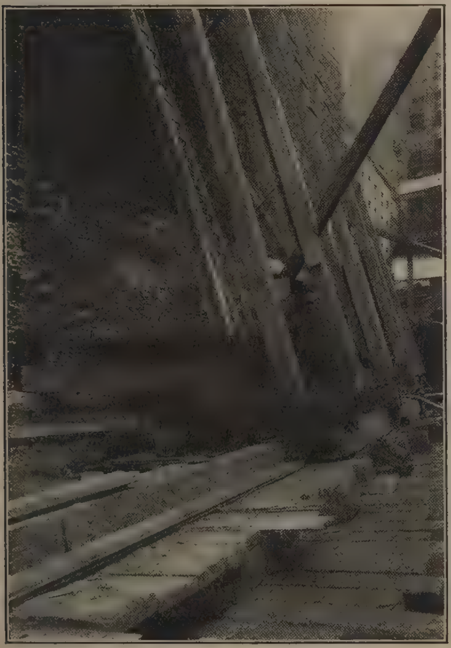
## Underpinning Adjacent to the Silversmiths' Building, New York.

The Silversmith's Building, which fronts at Nos. 15, 17 and 19 Maiden Lane and at Nos. 18 and 22 John St., is a 20-story steel cage office building, with a 75x75-ft. main part reaching through from one street to the other, and two 25x75-ft. wings 12 stories and 5 stories high, respectively, one of them containing an arcade connecting the two streets. The site was previously occupied by brick buildings, with their basement floors at an average level of about 12 ft. below the highest point of the curb. As soon as these were removed an inclined plane was built with timber falsework and planking, and on it was removed the spoil provided by excavating down to a general level of 20 ft. below the curb, except under the boiler room, the floor of which was at a depth of 22 ft.

As the foundations for all but one of the adjacent old buildings were carried to a depth of more than 20 ft. below the street level, this excavation did not interfere with their stability and was made without reference to the future operations. The ground water level at the site is about 22 ft. below the curb, and the rock is covered with sand and quicksand, on which the new grillage foundations are seated at a depth of from 25 to 28 ft. below curb. As this depth was greater than that of the footings of any of the adjacent old buildings, it was necessary to provide against undermining them or causing settlement before work on the new foundations was commenced.

A 20x50 ft. plank platform at street level, supported on falsework bents set in the bottom of the excavation, was built on the south side of the lot and afforded entrance for wagons from Maiden Lane and provided for loading and unloading materials there without obstructing the narrow, crowded street. On this platform was seated a stiff-leg derrick, with a 40-ft. boom, operated by a 2-drum Lidgerwood hoisting engine, with capstan heads, which were used to swing the mast by means of an ordinary bull wheel. The cause-way was removed, and the derrick was used for hoisting soil excavated from the pits for the foundation piers, which was shoveled into 1-yd. steel buckets and dumped in wagons driven on the working platform. Beams, girders, blocking and other supplies for the underpinning were similarly delivered to and from the excavation by the same derrick.

The two John St. wings are separated by the 5-story and basement dwelling at No. 20 John St., which is enclosed on three sides by the new building. Its exterior brick walls had footings carried down to a depth of about 17 ft. below the curb and were underpinned by new brick and concrete walls about 9 ft. deeper. The old walls were supported in the ordinary manner on needle beams made with pairs and double pairs of 12x12-in. timbers 25 ft. long, arranged two high and carried on jack screws at both ends. A line of full length timber sills and cribbing was placed across the cellar floor from end to end of the old building on the center line and supported the inner ends of the row of needle beams installed for the west wall, which was the first one underpinned. The outer ends of these beams were supported on blocking, or solid timber crib work, carried down in pits below the level of the new cellar floor.

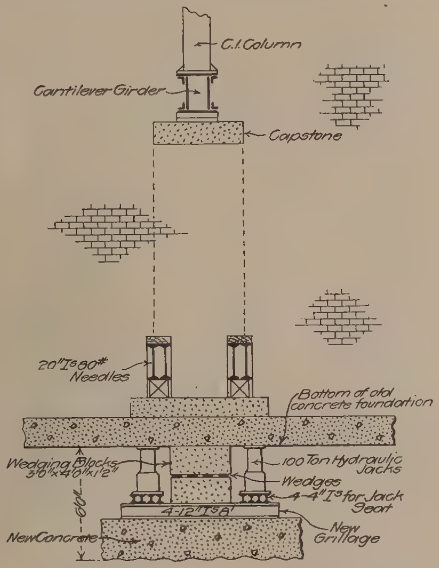


**General View of Shoring.**

While the underpinning of this small building was in progress work was commenced on the protections of the tall buildings on both sides of the lot. The west side of the lot is bounded by a 12-story and a 15-story steel cage building, with grillage foundations for their wall columns, respectively 22 ft. 8 in. and 24 ft. 2 in. below the curb. These structures were in good condition and only required to be safeguarded against undermining through movement of the sand below the surface, due to deeper adjacent excavations. To provide against this possibility they were protected by a continuous line of United States steel sheet piles 12 ft. long, with their upper ends driven down to a level of 1 ft. above the bottom of the grillages before the new foundation excavations were commenced.

The piles were driven easily and rapidly by a 1,200-lb. hammer, with leads 28 ft. high, framed to a light portable T-shaped timber bracing like that used for driving sheet piles in the Title Guarantee & Trust Company's Building on Broadway, less than 100 ft. distant from this building and similar to the device illustrated in The Engineering Record of August 27, 1904. The hammer was made special to project beyond the face of the guide so as to work close to the plane of the wall, and with it the piles were driven 3 in. from the brickwork. The line of piles was made continuous from one street to the other, and in one case a pile soon after being started became jammed in the joint with the last driven pile and carried it down with it far below the surface of the excavation. Fortunately this occurred at a point between the two buildings, where no special external pressure was to be apprehended, and the gap thus formed in the wall was left unclosed, without any evil results. The tops of the steel sheet piles on the west side of the lot were braced by horizontal transverse 12x12-in. timbers about 10 ft. apart, abutting at one end against a ledger piece bearing across the faces of the piles and at the other end were wedged against the longitudinal sills supporting the feet of the spur piles for the side wall of the Hayes Building. The thrust of these braces against the sill was taken up by the pressure from the inclined shores and by contact of the sills against the sheeting of the crib pit for the needle beams.

On the Maiden Lane side of the lot the front wall of the sidewalk vault, being outside of the new vault line, was allowed to remain undisturbed and was secured by inclined braces against a horizontal ledger piece, while short sections of wooden sheet piles were driven below its foot to retain the earth below it when the excavation was carried to a greater depth.



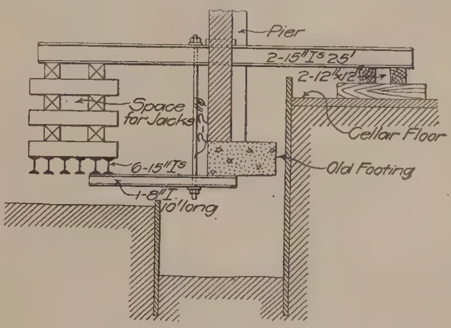
**Transferring Load to New Footing.**

On the John St. side of the lot the Cornell patent cast-iron combined curb and gutter, laid about 50 years ago, was found in a very good state of preservation and was temporarily left in service.

On the east side of the lot the John St. Building, five stories and basement in height, was supported in the usual manner on steel needle beams carried by wooden cribbing sills and timber piers, while the brick walls were carried down about 7 ft. further, or 25 ft. below the curb. On the same side of the lot the 8-story and cellar Hayes Building, at No. 21 Maiden Lane, is about 73 ft. deep and has cast-iron wall columns seated at street level on 4x4-ft and 4x5-ft. brick piers, with granite caps. The columns are enclosed in brickwork and connected by a 16-in. curtain wall ex-extended at the base to a thickness of 4 ft. and seated on a concrete footing about 4½ ft. wide and 2 ft. thick at the height of about 22 ft. below curb level. At the beginning of operation this wall was braced with sixteen 12x12-in. inclined shores, with their upper ends gained about 12 in. into the brickwork. The lower ends were seated with wedge adjustments on a continuous line of double 12x12-in. horizontal timbers parallel to the wall and about 12 ft. from it, which was supported on a continuous grillage about 6 ft. wide of timbers. Ten of the shores were 40 ft. long and six were 25 ft. long. Three similar shores were provided to brace the exposed face of the rear wall column, to which were bolted three cast-iron lugs to receive the upper ends of the shores. The front wall of the building was braced by one 40-ft. and one 50-ft. inclined shore, with their feet outside of the sidewalk.

Opposite the 5-story building at 20 John St. pits 5 ft. wide and 8 ft. long were sheeted down about 10 ft. to a level below that of the foundation excavation, and in them were built timber crib work to support the ends of the needle beams, which carried the walls of this building and of the one at No. 24 John St. Sixteen other pits about 6 ft. square and 8 ft. deep were sheeted down to carry the crib work for other needle beams below the foundation excavation. Eight of them were located about 6 ft. apart in a row parallel with the side wall of the Hayes Building and 4 ft. in the clear away from it. These pits were sheeted with 3x10-in. tongue and groove yellow pine driven in the manner described in the recent article illustrating the work at the Murray Street Building by an Ingersoll air drill operated by steam purchased from the New York Steam Co.

At each corner of the pit the two adjacent units of sheeting were lag screwed to a 6x6x⅜-in. vertical angle, which insured the alignment and water tightness of the excavation. The sheeting was braced against 4x10-in. waling pieces, and the excavation was carried down with pick and shovel as the sheeting was driven. Considerable water was encountered, and in most of the pits it was removed by diaphragm pumps. In the deeper pits these were found inadequate, and an Edson double diaphragm pump was rigged with a walking beam and operated by a gasolene engine. Concrete footings were built in these pits and in most cases consisted of 2 ft. of 1:2½:5 hand-mixed concrete shoveled in. In some cases the water was so troublesome that it was necessary to deposit the concrete in bags, one of which was placed as fast as each shovelful of earth was removed. In each pit a 5x5-ft. pier of solid 12x12-in. yellow pine timbers was built up from the concrete footings to about the level of the top of the footing of the wall of the Hayes Building and supported a continuous line of longitudinal stringers 5 ft. in the clear from the wall, which was composed of six parallel 15-in. 60-lb. I-beams. These stringers supported the outer ends of 17 needle beams, each of them composed of 20-in. 80-lb. I-beams 25 ft long, with their inner ends resting on longitudinal double 12x12-in. continuous timbers about 5 ft. in the cle from the wall, which were laid on a continu grillage 6 ft. wide of transverse 12-in. tim laid on the cellar floor. Special needle were used at the corners of the building; the intermediate ones consisted of two each and were supported by one 10-ton j under each end of each I-beam, making each regular needle. These were set I-beams had a clear span of 14 ft. b



**Suspended Support for Footing.**

inner jackscrews and were calculated to have a maximum load of 50 tons each.

The cast-iron wall columns of the Hayes Building, being eccentric from their piers, are seated on cantilever girders, which have rocker bearing on cast-iron pedestals seated on capstones built into the solid brick work of the pier integral with the wall several feet above the old concrete footings. These footings were in such good condition that it required over 150 hours' work to trim a length of about 10 lin. ft. that had been partly broken away by accident. It was therefore decided, both for economy of material and more especially for the greater rapidity, to retain the old footings and build the new foundations about 6 ft. high under them. Two sets of needle beams were accordingly inserted through the old wall directly under the pedestal supporting each column and at a height just sufficient to give a working clearance over the old concrete footings.

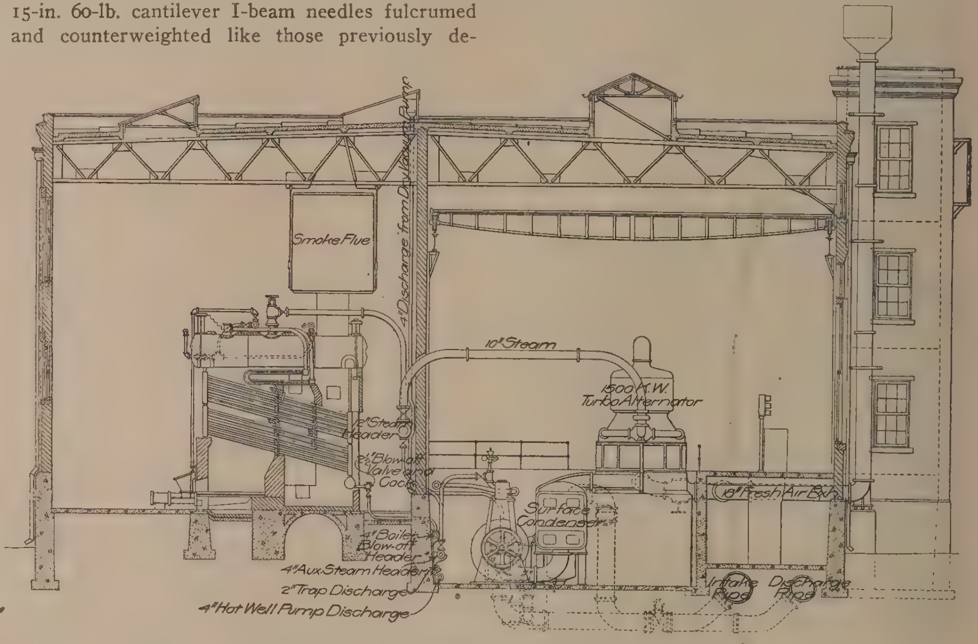
A steel bearing plate was seated across the top flanges of each pair of I-beams and gave bearing for the nuts on the upper ends of two 2-in. vertical rods about 7 ft. long. The nuts on the lower ends of these rods engaged a cross plate or saddle, forming a fulcrum for an 8-in. horizontal cantilever I-beam 10 ft. long. The long arms of the cantilever reacted upward against some of the I-beam stringers supporting the outer ends of the needle beams. The short arms took bearings about 2 ft. long on the under side of the old concrete footing, supporting it across the thickness of the wall, so that when undermined by the excavation for the new foundation the old footing looked in cross sections like a cantilever projecting about 2 ft. beyond the inner face of the wall and proved strong enough to resist the bending moment thus developed. The wall columns were about 9 ft. apart and the footing was divided in sections of corresponding length, which were alternately supported, undermined and underpinned, and were transferred to the new footings before work on the adjacent sections was commenced.

The new concrete footings were built in alternate sections 9 ft. long, excavated on the centers of the wall columns. The upper surface of the new footing was leveled off about 3½ ft. below the lower surface of the old footing, and on it on the center of each column was set a grillage of four 12-in. longitudinal I-beams 8 ft. long thoroughly bolted together with separators and concreted. In the middle of each footing was set a pair of 3x4-ft. cut granite blocks, 14 in. thick, separated by pairs of thin steel wedges. On each end of the grillage four transverse 4-in. I-beams were placed to form a seat for a 100-ton hydraulic jack reacting against the bottom of the old foundation.

As the jacks were operated the wedges were driven to maintain solid bearing between the old and new footings, with the granite blocks on the column center and thus prevented the possibility of displacement or injury to the wall from any failure of the jacks. The entire weight of wall and column was thus transferred to the new foundation, and the latter was rapidly forced to final bearing, so that no further settlement took place when the jacks were removed. By this method it is probable that excessive proportions of the loads were temporarily concentrated on each section of the footing, thus fully testing them and compressing the soil in excess of work- stress and insuring against future settlement. r the jacks were removed additional pairs dging blocks were inserted in the positions d by them, and the remaining spaces be- the old and new footings were closed in l manner.

the columns were placed close together ediate needle beams were required be- piers, but where the space was greater it was sub-divided by pairs of intermediate needle beams. The loads on the intermediate column piers were assumed at 200 tons each, but at the Maiden Lane end of the wall the pier at the corner of the building carries part of the front of the building and has an estimated load of about 300 tons. It was, therefore, carried by two regular needles, together containing four I-beams, and by one special needle, containing four more I-beams, making eight in all.

At the other end of the wall there is, adjacent to the corner pier, a 3x3-ft. pier, carrying a 24-in. iron smokestack 100 ft. high, with an estimated weight of about 6 tons. This was supported during underpinning by a single cantilever needle beam made with one 12x12-in. yellow pine timber, 25 ft. long, fulcrumed on the I-beam sills, which support the outer ends of the regular needle beams and counterweighted at the extremity of the 25-ft. long arm by a 1-yd. steel bucket filled with earth. Beyond this smokestack there is a short length of 12-in. 2-story brick wall, which during underpinning was supported by two 15-in. 60-lb. cantilever I-beam needles fulcrumed and counterweighted like those previously described. The end of this wall intersects a 20-in. cross wall, supported at the corner by an oblique cantilever needle beam made with a single 12x12-in. yellow pine timber 25 ft. long, fulcrumed on the last mentioned cantilever needle beam and anchored at the outer end by a wire cable attached to the blocking under the regular needle ebam. The jackscrews were operated to give all of the needle beams a deflection of about ¼ in. at the center and carry the full weight of the wall without causing any appreciable displacement. The old footings were then removed, the trench dug down to a depth of 1 ft. below the deepest point of the new foundations and the new footings and brickwork above them built and the weight of the old wall transferred to them in the usual manner. Careful levels were taken on a bench mark on the opposite side of the street, and repeated observations were made, which failed to show any settlement of the old walls during or after the underpinning process.


Smoke Flue
4" Discharge from Dry Vacuum Pump
10" Steam
1500 K.W. Turbo Alternator
12" Steam Header
2½" Blow-off Valve and Cock
4" Boiler Blow-off Header
4" Aux. Steam Header
2" Trap Discharge
4" Hot Well Pump Discharge
Surface Condenser
18" Fresh Air Exh.
Intake Pipe
Discharge Pipe

Cross-Sectional Elevation of Whitewood Station.

Nearly the full width of the Maiden Lane front of the Hayes Building is occupied by three large openings for the main entrance and wide show windows. One of these is spanned by a very flat stone arch, with the skewbacks built into the corner pier of the building very close to the underpinned side walls. To avoid the possibility of displacement of these side walls during the above described operations a pair of long inclined 12x12-in. timbers were wedged against the face of the side wall opposite the skewback of this arch, and their feet were made to engage the vault retaining wall just below sidewalk level. A similar pair of horizontal timbers were wedged against the wall at first floor level, and the two sets were united by vertical and diagonal bracing planks spiked on to them, making a sort of truss and thoroughly providing against any possibility of movement due to the thrust of the arch. As these timbers required wedging again after the work was commenced, it was assumed that they did not actually receive much stress and that their use was a precaution which might have been safely omitted.

Excavation below the cellar floor of the old building was commenced July 4, 1907, the underpinning contract was awarded July 12, and work on it was commenced the same day and carried on night and day, with two gangs of about 20 men each. On August 1 the underpinnings of the buildings at No. 20 and at 24 John St. were completed, the first pier was built in its pit on August 8, and on August 24 the underpinning of the Hayes Building was substantially finished and the work practically completed.

Clinton & Russell are the architects of the building, and the general contractor is the A. J. Robertson Co., Mr. L. H. Whitenack, superintendent in charge. The contract for underpinning was awarded to Miller, Daybill & Co. and was executed under the personal supervision of Mr. Alfred Daybill.

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Patented Pavements were the subject of a recent decision of the Louisiana Supreme Court in Lacoste v. City of New Orleans, 44 S. Rep. 267. The court's syllabus of the decision is as follows: The fact that a pavement is patented is no obstacle to full and fair competition upon a municipal contract for the laying of it, when the patentee has filed with the city authorities an agreement to let the successful competitor for the contract have the free use of the patent upon payment of a fixed royalty, thereby placing all prospective competitors upon an equal footing; it appearing that the royalty thus exacted was reasonable, and did not destroy the margin of profit under the contract. It is no answer to say that the patentee can underbid his competitors for the contract. To him, as to the other bidders, the sole inducement for bidding on the contract is the margin of profit which he sees in it, and for him, as for all others, this margin of profit begins only after the royalty has been paid.

## A Central Power Station in the Whitewood, S. D., Mining District.

The generating station of the Consolidated Power & Light Co. of Deadwood, S. D., has been built to supply electric current for power in the mines and mills in its vicinity, and for lighting in Deadwood, Lead and several smaller mining camps. This company was organized as a consolidation of the Belt Light & Power Co., and the Deadwood Electric Light Co. The station of the former company was at Pluma, midway between Deadwood and Lead, on Whitewood Creek and the Burlington R. R. This station was so favorably located that the decision was made to reconstruct and enlarge it according to the best practice, and to abandon the Deadwood station.

The station building as reconstructed and enlarged is 88 ft. wide and has an extreme length of 136 ft., as shown in the accompanying general plan. It is divided by a longitudinal brick wall into a boiler room, 40 ft. wide, and a generator room, 44.5 ft. wide. The original station contained two 200-kw. DeLavel turbo-generator units which were supplied with steam by three 200 h.-p. Freeman Scotch boilers. These generators and boilers have been left in their original position in the old part of the building, but have not been in service since the new equipment was installed.

The boiler room contains four new Babcock & Wilcox boilers, arranged in two batteries, with space at the end of the row for two additional boilers. These boilers each have 3,500 sq. ft. of heating surface and are equipped with superheaters capable of raising the steam temperature 100° Fahr. above that of saturated steam. The furnace of each boiler is equipped with a Jones underfeed stoker, the forced draft used with the stokers being furnished by two American Blower Co. 120-in. blowers, either of which has sufficient capacity to supply draft to all four furnaces. These blowers and the two engines driving them are placed in a small separate room at the rear of the boiler room. The stoker rams are operated by steam in the usual manner, the automatic device controlling the supply of steam being driven by either of the blower engines. The speed of the latter is controlled by a Foster regulating valve arranged in such manner that the coal and air supply are varied automatically, so the boiler pressure is maintained constant within certain limits.

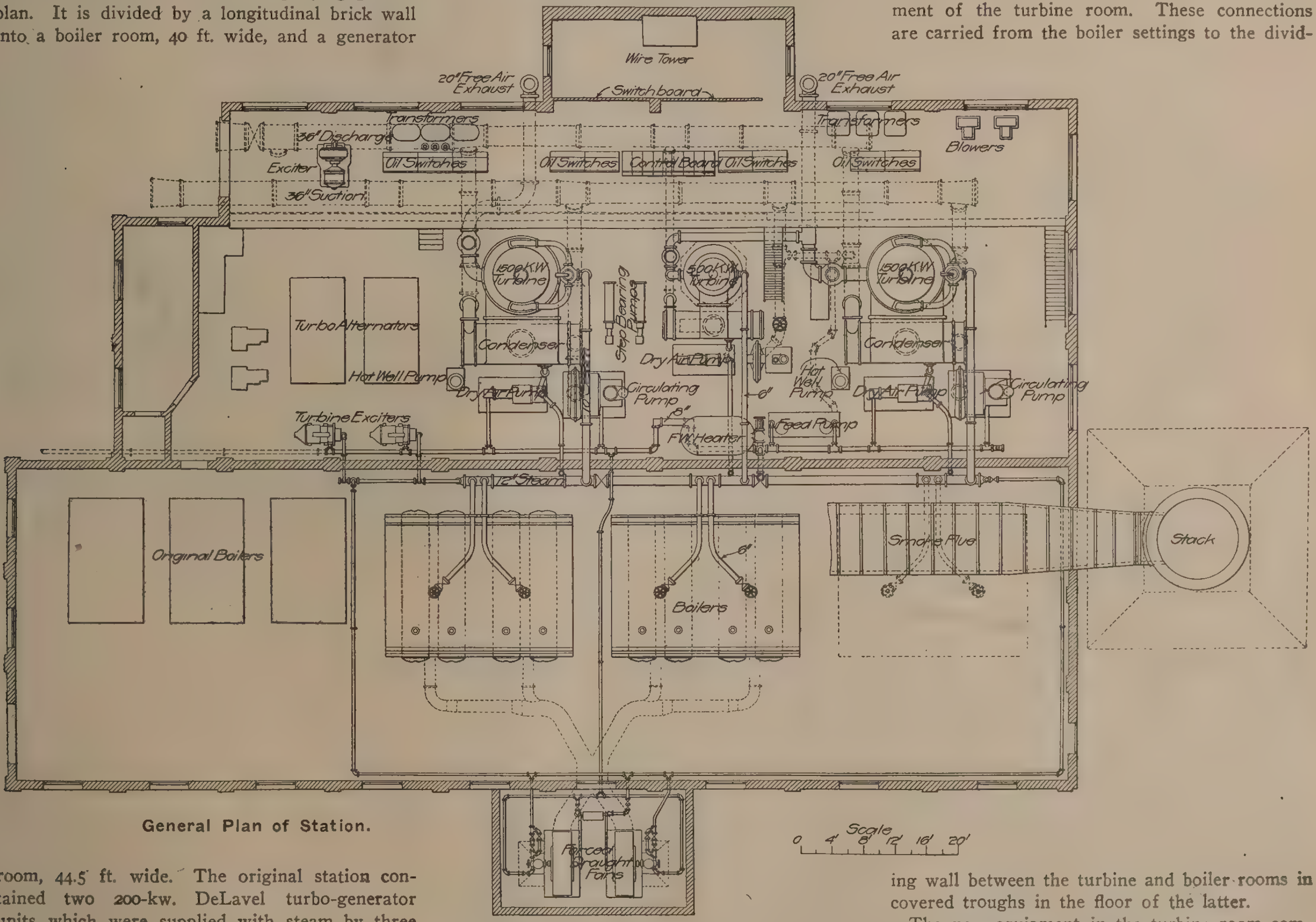
Wyoming lignite containing about 11,000 B. t. u. per pound when dry, is burned under the boilers; as this lignite is used at present, however, it contains approximately 20 per cent. of moisture. A temporary coal-handling arrangement is employed, pending the installation of a proposed coal-handling system. Coal is delivered one end of the building at present and then hauled into the boiler room and stored on the firing floor, from which it is supplied to the stokers by hand. A large structural steel storage bunker is to be erected along the rear side of the boiler room. A switch track will be extended over this bunker, in order that coal may be dumped into the latter from the cars. Chutes will lead from the bunker directly to the charging hoppers of the stokers, thus permitting the latter to be fed by gravity throughout.

Feed water is supplied to the boilers through duplicate feed lines by either of two vertical marine-type single-cylinder Dean Bros. boiler-feed pumps, either of which has sufficient capacity to furnish water to all four boilers under any operating conditions. These pumps draw from an open-exhaust Stillwell feed-water heater in the turbine room to which water is pumped from a hot-well in the basement of the generator room. Make-up water is supplied automatically by a low-service Blake pump from a circulating system for condensers in the turbine room. A float-regulated valve in the discharge of this pump actuates a Fisher governor which controls the operation of this pump. The main units in the turbine room are all operated condensing, the water of condensation passing to the hot well. The auxiliary steam-consuming units in the station are all run non-condensing, their exhaust being carried to the feed-water heater.

The boilers each have a 6-in. connection with a 12-in. steam header along the wall in the rear of their settings. These connections are each equipped at the boiler with a stop and a check valve. The header is divided in three parts, one for each pair of boilers, by stop valves. Each of these three parts is connected with a 4-in. auxiliary steam header in the basement of the turbine room, forming a ring system which enables the main units in that room to be supplied with steam under practically any conditions.

The mud drum of each boiler has two connections with a 4½-in. blow-off header in the basement of the turbine room. These connections are carried from the boiler settings to the dividing wall between the turbine and boiler rooms in covered troughs in the floor of the latter.

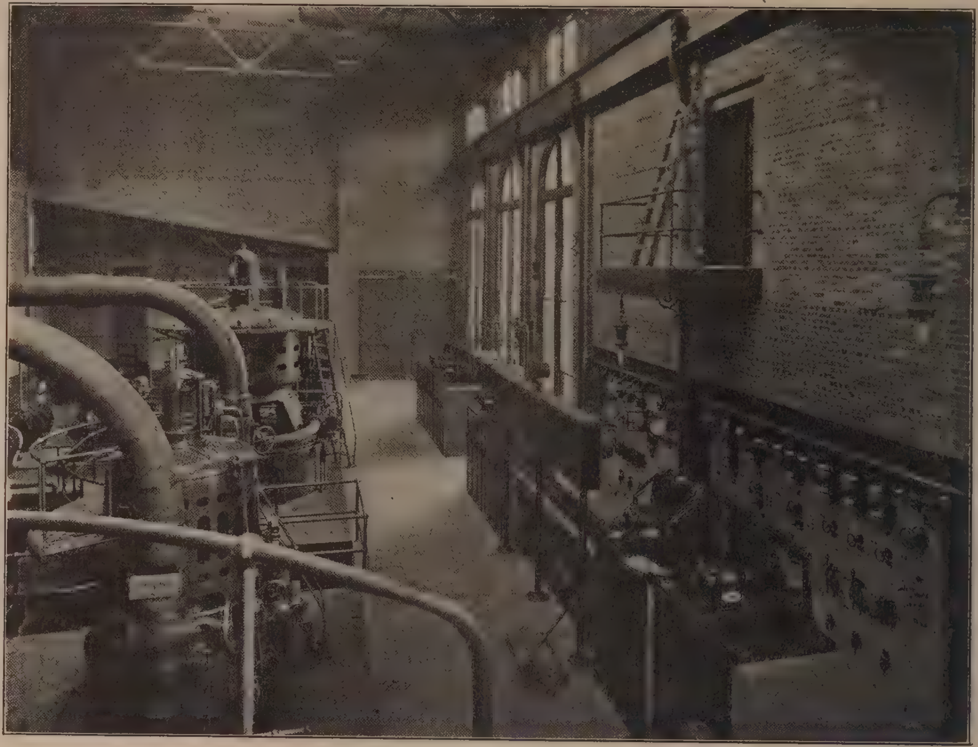
The new equipment in the turbine room comprises three main turbo-generator units together with their necessary auxiliary apparatus. One of these units is a 500-kw. 2,300-volt 60-cycle three-phase General Electric generator, direct-connected to a four-stage Curtis turbine, with water supported step bearings. The other two units each have a 1,500-kw. 11,000-volt 60-cycle three-phase General Electric generator direct-connected to a four-stage Curtis turbine; the turbines of these units also have water-supported bearings. The 500-kw. unit has a Wheeler admiralty condenser with 2,000 sq. ft. of surface, equipped with a 6x14x10-in. dry-air pump and a 10-in. steam-driven horizontal-suction centrifugal pump for circulating cooling water. The two large units also have condensers of the same type, with 6,000 sq. ft. of cooling surface, which are each served by an 8x18x12-in. dry-air pump and an 18-in. steam-driven horizontal-suction centrifugal pump for the cooling water. The exhausts of all three



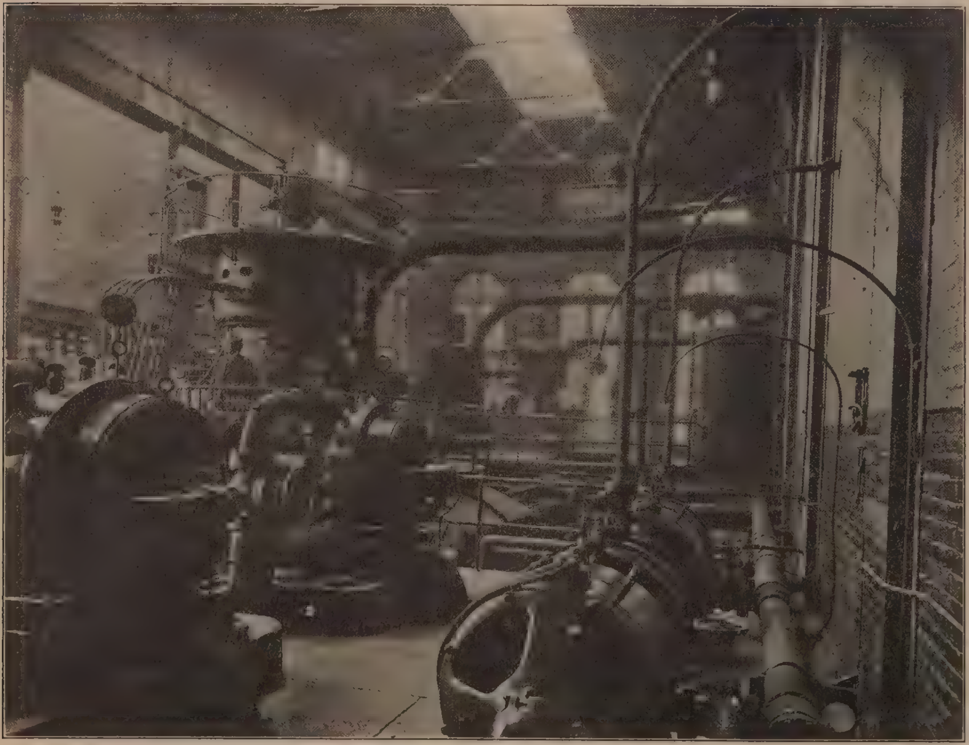
General Plan of Station.

units are connected to risers extending above the roof of the building where they are fitted with Wright exhaust heads. A 42-in. reinforced-concrete conduit extends from the channel of Whitewood Creek, which is about 200 ft. from the station building, to the latter and is continued into the building as a 36-in. main from which the circulating pumps take suction. The cooling water is discharged from the condensers into a second 36-in. main beneath the floor of the basement of the turbine room; this main terminates at the building line in a second 42-in. reinforced-concrete conduit leading back to the creek.

Owing to a scarcity of water in the creek at certain seasons, a dam has been erected across the channel of the latter, and a specially devised cooling system has been added to the end of the discharge conduit. This cooling system is only used when there is an insufficient quantity of water coming down the creek. It consists essentially of a manifold of standard hub-and-spigot cast-iron pipe, ranging from 24 in. to 16 in. in diameter, with special crosses having 5-in. flanged outlets for lateral pipes fitted with a series of sixty 3-in. Schutte spray nozzles. The manifold pipe is carried by concrete piers; the lateral pipes are each supported at the middle by a truss rod equipped with a turnbuckle and attached to a strut on the manifold. This arrangement has entirely prevented vibration. The laterals are made up of 5-in. and 3-in. pipe, with a total length of 22.5 ft. from the center of the manifold to the center of the outer nozzle, two of which are carried by each lateral. The nozzles are spaced 15 ft. apart on centers in all directions. Each nozzle is designed to spray 9,000 gal. of water an hour under a pressure of 12 to 15 lb. per square inch, but two valves in the manifold permit part of the nozzles to be cut out of service. The complete distribution of the spray by this method is shown in one of the illustrations. The dry air of the locality causes sufficient evaporation from the spray to cool the water satisfactorily.



Switches and Switchboards in Turbine Room.



Three Main Units in Turbine Room.

Three exciter units for the main generators are provided. Two of these each have a 25-kw. 125-volt direct-current generator driven by a steam turbine. The third is a 60-kw. 125-volt machine direct-connected to a 75 h.-p. three-phase 2,300-volt induction motor. The steam-driven exciters are against the rear wall of the turbine room; the motor-driven exciter is on a concrete platform at the front side of that room and level with the floor of the latter.

The switches, transformers and so forth are on the concrete platform along the front side of the generator room. Two switchboards are provided, one for the 2,300-volt circuits and the other for 11,000-volt. Three single-phase 2,300-volt lines are carried from the 2,300-volt board to Deadwood and three to Lead for commercial lighting. Two circuits for street arcs are also carried to Lead and one to Deadwood from this board. The exciters are also controlled from the latter.

The two 1,500-kw. units and the 11,000-volt transmission lines are equipped with motor-operated three-phase oil switches. These motors are controlled from the second switchboard by the remote control system can be operated at voltages from 70 to 150 volts, the current being obtained from the exciter units. The transmission line switches can also be operated by hand in an emergency.

The 11,000-volt bus bars are in separate brick and concrete compartments in the space under the concrete platform carrying the oil-switches and transformers. The generators are connected to the ends of the bus bars, and disconnecting switches are provided in the latter between the generators and the circuits leads which are taken off from the middle portion of the buses. Disconnecting switches are also placed between each motor-operated oil switch and the bus bars for safety during repairs and cleaning.

Two banks of transformers are arranged between the two switchboards, in order that current may be taken from either one to the other. One set consists of three 100-kw. oil-cooled type, and the other of three 185-kw. air-blast type.

A fireproof brick tower, 10x30 ft. in plan and three stories in height, is provided on the front side of the building for all transmission lines leaving the station. The 2,300-volt switchboard is placed in an opening in the building side of the first story of this tower, and together with its connections and equipment occupies most of that story although very ample room for reaching the back of this board is available. Current regulating transformers for the 2,300-volt lighting transmission lines occupy the second floor, and lightning arresters, choke coils and so forth, the third. The 2,300-volt lines in the tower are oiled-cambric insulated cable; the 11,000-volt lines are bare wire in separate brick cells and on 20,000-volt insulators.

The 2,300-volt transmission lines to Deadwood and to Lead are both 1½-miles long. The 11,000-volt lines extend to various mining camps within a radius of 10 miles of the station. None of these lines embodies any special features of construction

Mr. N. E. Franklin is vice-president and treasurer, Mr. J. W. Springer, manager, and Mr. J. H. Fuller, engineer, for the Consolidated Light & Power Co. Messrs. Sargent & Lundy, of Chicago, are consulting engineers for that company and were represented during the construction of the station by Mr. H. Boyd Brydon, to whom this journal is indebted for the information from which the foregoing notes were prepared. The extensive piping work in connection with the station was done by Mr. W. H. Pope, of Chicago.

The Spraying System of the Whitewood Power Plant in Full Operation and under Light Pressure.

## Continuity in Reinforced Concrete Beams and Slabs.

The question of rejecting or recognizing continuity in the design of reinforced concrete slabs and beams is one of the disputed features of such work that will probably not be settled for a long time. The continuity is necessarily assumed when it is assumed that beam and not arch action takes place. This assumption is generally considered to be correct except where the ratio of depth to span is great, but it is possible that more attention will be given to arch action when experiments have become more numerous and comprehensive. The subject was discussed by Prof. Lewis J. Johnson of Harvard University at a meeting of the Boston Society of Civil Engineers some time ago, and the necessity of providing for continuous beam action was strongly urged in his remarks. Slabs, beams and girders are continuous beams as built, and for better or for worse will act as continuous beams, in his opinion, if they act as beams at all. Objection to recognizing this fact in design seems to be based upon the supposition that designing beams as continuous would lead to mid-span sections good only for positive bending moments of about $1/40\ wl^2$, a figure which would be nearly correct for a load distributed uniformly over all spans at once, but would be fatally in error in the case of the far more probable instances of unequally distributed load.

Objection of a similar sort is applicable to design upon the basis of discontinuity, where ultra cautious design for a mid-span bending moment of $\frac{1}{8}\ wl^2$ is accompanied by a more or less complete ignoring of the negative bending moment of the supports. The assumption of simple beam action may thus prompt serious error on the side of danger quite as certainly as similarly improper application of the assumption of continuous action. That trouble from this source has not been more abundant Prof. Johnson believes to be due partly at least to the undoubtedly considerable tensile strength of the concrete in the wings of T-section beams and girders. But if it is not safe to count on tensile strength of concrete elsewhere, it is not safe to count on it here, he holds.

The logical course to pursue, he states, is to recognize that, if the action present is beam action at all, it is continuous beam action, and to design accordingly. This means careful attention to the extreme values of the flexures at supports as well as at mid-spans due to all possible distributions of the live load. The end sections are then designed to meet these extreme conditions, and so are the mid-span sections. This is standard practice among the Germans and Swiss. Of course in applying this method, as in any careful design, attention will be given to all important facts, such as stair or elevator wells interrupting the continuity in places; and in cases of doubt, assumptions unmistakably on the side of safety will, of course, be made.

The labor involved in these computations is not so great as it would seem. As a matter of fact, the extreme conditions under uniformly distributed live load will almost always be covered in case of a series of beams and slabs of equal spans by designing for a live load flexure of $1/10\ wl^2$ ($l$ being measured from center to center of supports) at the faces of columns and girders, and the same amount at mid-spans; and this, too, regardless of the number of spans in the line. At the column faces flexure would be negative and at mid-spans positive, and in both cases would, of course, be combined with the dead load flexures. If the spans are short and the live loads large in comparison to the dead, top reinforcement may be required at mid-span to provide for resultant negative flexures there existent. The similar extreme values for girders subject to concentrated loads have not been so well established, but the need of them is recognized and it is hoped that they may be forthcoming soon.

If a designer prefer, let him use $\frac{1}{8}\ wl^2$ at the mid-span section, but let him not fail to provide fully for the bending at the faces of the supports, Prof. Johnson advises. He must not overlook the fact that top reinforcement is as logical a requirement as bottom reinforcement at mid-span. Moreover there is additional justification for top reinforcement in that it is of the greatest possible value in case of weakening of bottom rods by fire. The top rods through cantilever action may carry the load after the lower rods, in the far more exposed position of the two, have failed. In fact, top reinforcement does not seem to have had the attention which its merits from the fireproofing standpoint would seem to entitle it.

It may be objected that continuous beam coefficients based upon the assumption of unvarying moment of inertia may be inapplicable to reinforced concrete beams. This is certainly a fair field for research, but Prof. Johnson believes that the practitioner may well proceed for the present with his $1/10\ wl^2$, taking comfort from realizing that the negative bending moment over supports would not rise above $\frac{1}{8}\ wl^2$ even in the extreme case when the moment of inertia becomes zero at mid-span, the case of two abutting disconnected cantilevers, most unlikely to occur. The error in the $1/10\ wl^2$, if any exist, must be extremely small and unimportant.

For beams under a uniformly distributed load, the top steel at the column faces and through the column may or may not be the same in amount as at mid-span, depending upon the relative depth of the beam at the two points, but Professor Johnson sees no escape from the belief that the negative moment of resistance at the column face should be as large as the positive moment of resistance at mid-span. If, as is usual, floors are figured with T-sections, this may call for the German practice of materially deepening the stems at, and for considerable distances each way from, the supports, to make up for the absence of flanges on what is here the compression side of the beam.

This leads to brackets at connections of girders to columns and of beams to girders. These brackets complicate the forms and are sometimes unsightly. They can be obviated by making the depth of stem at mid-span as great as required at the column faces, proportioning the bottom steel accordingly. This interferes with headroom and adds to the quantity of concrete required. It may in some cases be practicable to diminish this depth by the use of steel reinforcement in the compression lower side of the beam at the support. Though this latter reinforcement would also be effective and necessary reinforcement from the arch point of view, the brackets will in many cases be preferred to either of these alternatives.

Even if all these considerations are kept in mind, continuous beam action is unavoidable, according to Prof. Johnson. It must be reckoned with and properly provided for in all reputable reinforced concrete practice, he asserts, for there must be no departure from the principle of providing at all points of a structure for the most unfavorable conditions it is reasonable to expect. Moreover, it will not do to assume that we err on the side of safety when we ignore continuous beam action.

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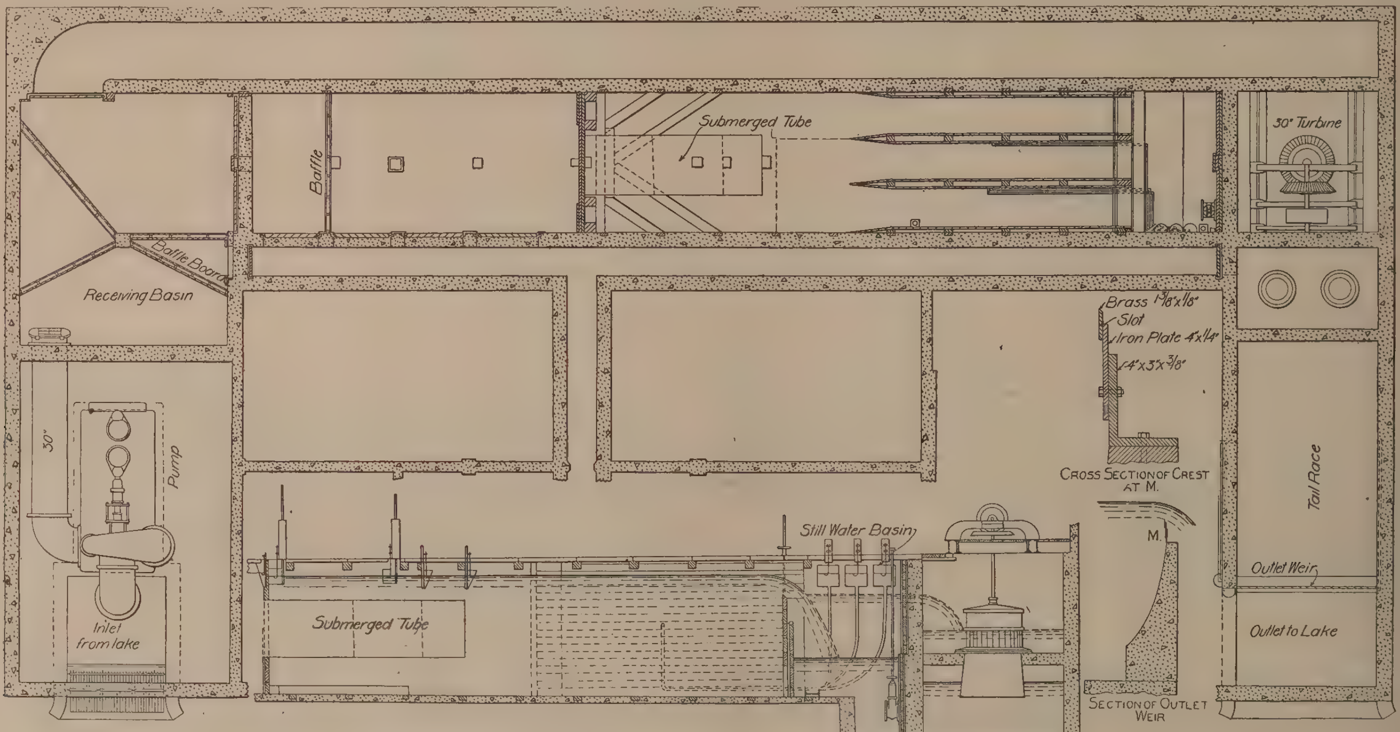
AN EXTENSIVE WIRELESS TELEGRAPH SYSTEM has been installed by the United Fruit Co. on its fruit steamers and at numerous points on the Gulf of Mexico and the Caribbean Sea, for communication between the offices, steamers in transit, and its plantations in Central America and the West Indies. A station was recently added at San Antonio at the extreme western end of Cuba.

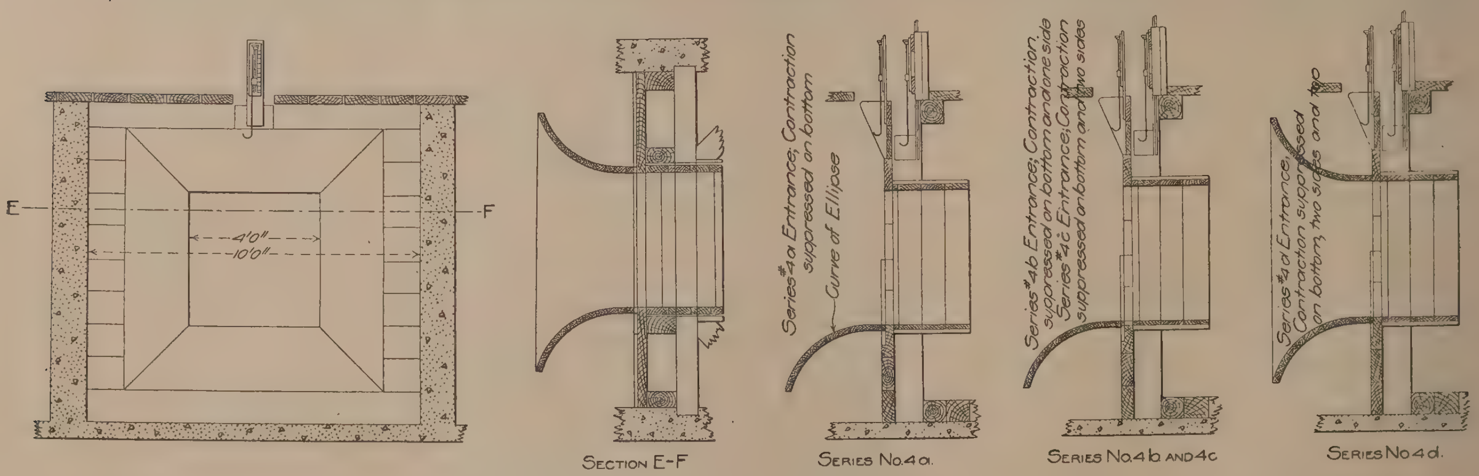
## Experiments With Submerged Tubes 4 Ft. Square.

By C. B. Stewart, in Charge of Hydraulic Research Work, University of Wisconsin.

The recent construction of large works in connection with the various lines of Hydraulic Engineering has emphasized the need of further experimental knowledge concerning the losses of head resulting from various forms of resistances interposed in flowing streams of considerable size. The loss of head resulting from the flow of water through submerged gates, tubes and orifices of considerable size and with various forms of entrance is one of the most important divisions of the general subject which needs experimental investigation. Practically the only reliable experimental data available concerning the flow of water through large orifices are given in a valuable paper by Mr. Theodore G. Ellis, entitled "Description and Results of Hydraulic Experiments with large Apertures at Holyoke, Massachusetts," and printed in the "Transactions" of the American Society of Civil Engineers for 1875. Mr. Ellis determined the loss of head for flow through apertures the dimensions of which were 2 ft. square and 1 x 2-ft. when discharging freely into the atmosphere, and also made a very few experiments on the loss of head for flow through submerged apertures 1 ft. square and 1 ft. in diameter under heads from 2 ft. to 15 ft. In nearly all the experiments of Mr. Ellis, the thickness of the edge of the aperture remained constant, about ½ in. for wood and ¼ in. for iron, and perfect contractions of the emerging stream were supposed to take place on all sides of the aperture. In the single case of the aperture 1 ft. square, the entrance was subsequently modified so as to have a curved approach on all four sides, and the resulting coefficients of discharge were determined.



Plan and Section of Apparatus for Testing 4-Foot Square, Submerged Tubes.


E
F
4'0"
10'0"
Series #4a Entrance; Contraction suppressed on bottom
Curve of Ellipse
Series #4b Entrance; Contraction suppressed on bottom and one side
Series #4c Entrance; Contraction suppressed on bottom and two sides
Series #4d Entrance, Contraction suppressed on bottom, two sides and top

Section E-F Series No. 4a. Series No. 4b and 4c Series No 4d.

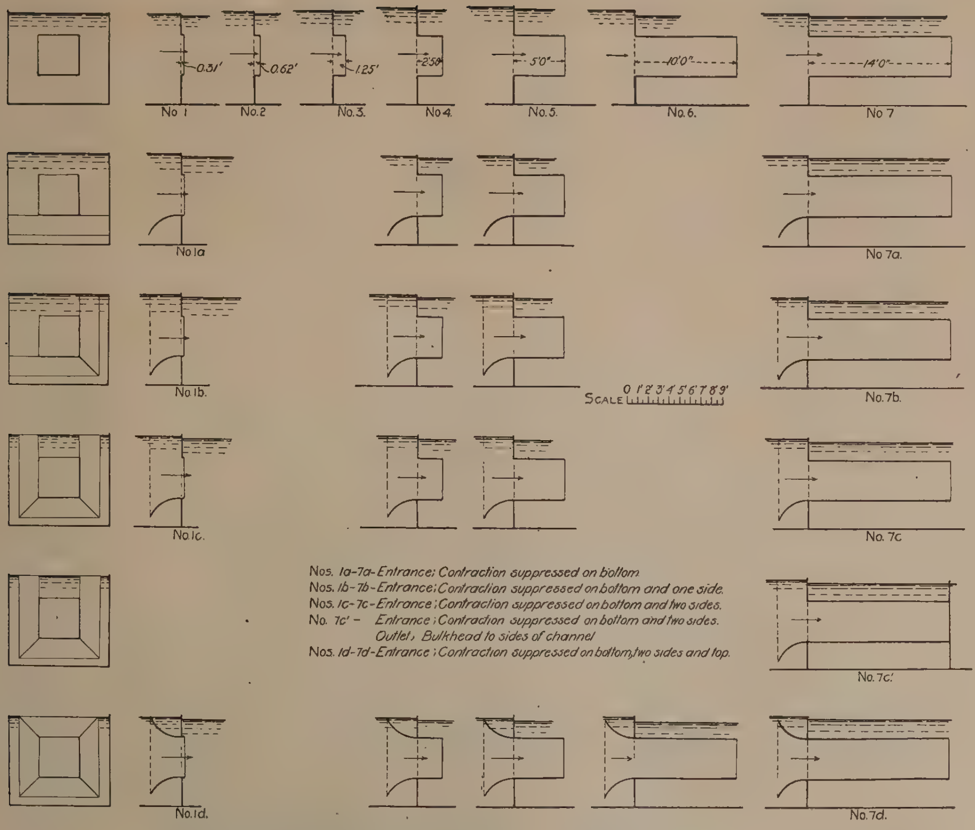
Details of Tubes with Suppressed Contractions.

In the experiments at the University of Wisconsin described in the present paper, the cross-section of the orifice or tube has been kept constant, being 4 ft. square, and the effects of changing the length of the tube and of modifying the entrance conditions by curved approaches on one, two, three and four sides respectively, for each of the various lengths, has been studied.

In future experiments on this subject at the University, it is planned to determine the coefficient of discharge for submerged tubes and orifices of various forms and areas of cross-section, with and without entrance modifications. It is hoped that the results of the experiments when completed will fairly cover the field of investigation laid out and be of material aid to the engineering profession.

The experiments were made at the new hydraulic laboratory at the University of Wisconsin (Engineering Record, April 21, 1906), which is located on the shore of Lake Mendota and furnishes an abundant supply of fresh water. Exceptional facilities were afforded for experiments on a large scale, and care was taken that errors of observation should be reduced to as small a percentage as possible. The accompanying plan of the laboratory shows the main features and arrangement for the experiments. The water was taken from the lake by a suitable inlet and raised about 12 ft. by means of a 30-in. Morris centrifugal pump, and delivered into a receiving basin which measures about 15 x 18 x 10 ft. deep. From the receiving basin the water flows through a flume 10 ft. wide, 10 ft. deep and about 68 ft. long, and may be used for operating a 30-in. turbine located at the end of the flume or may be wasted through gates underneath the turbine. The water flows back to the lake through a tail race 10 ft. wide, as shown. At the outlet end of the tail race is a sharp crested weir with end contractions suppressed and a crest 10 ft. long.

Suitable baffle boards were inserted in the receiving basin and at the entrance to the flume until it was found by trial that the water in the flume flowed in lines practically parallel to the sides of the flume.


0.31'
0.62'
1.25'
2.50'
5.0'
10'0"
14'0"
No 1
No.2
No.3.
No 4
No.5.
No.6.
No 7
No.1a
No 7a.
No.1b.
No.7b.
Scale 0 1 2 3 4 5 6 7 8 9
No.1c.
No. 7c
Nos. 1a-7a-Entrance; Contraction suppressed on bottom.
Nos. 1b-7b-Entrance; Contraction suppressed on bottom and one side.
Nos. 1c-7c-Entrance; Contraction suppressed on bottom and two sides.
No. 7c' - Entrance; Contraction suppressed on bottom and two sides. Outlet; Bulkhead to sides of channel
Nos. 1d-7d-Entrance; Contraction suppressed on bottom, two sides and top.
No. 7c'.
No.1d.
No.7d.

Sections of Tubes Showing Forms of Entrances and Lengths.

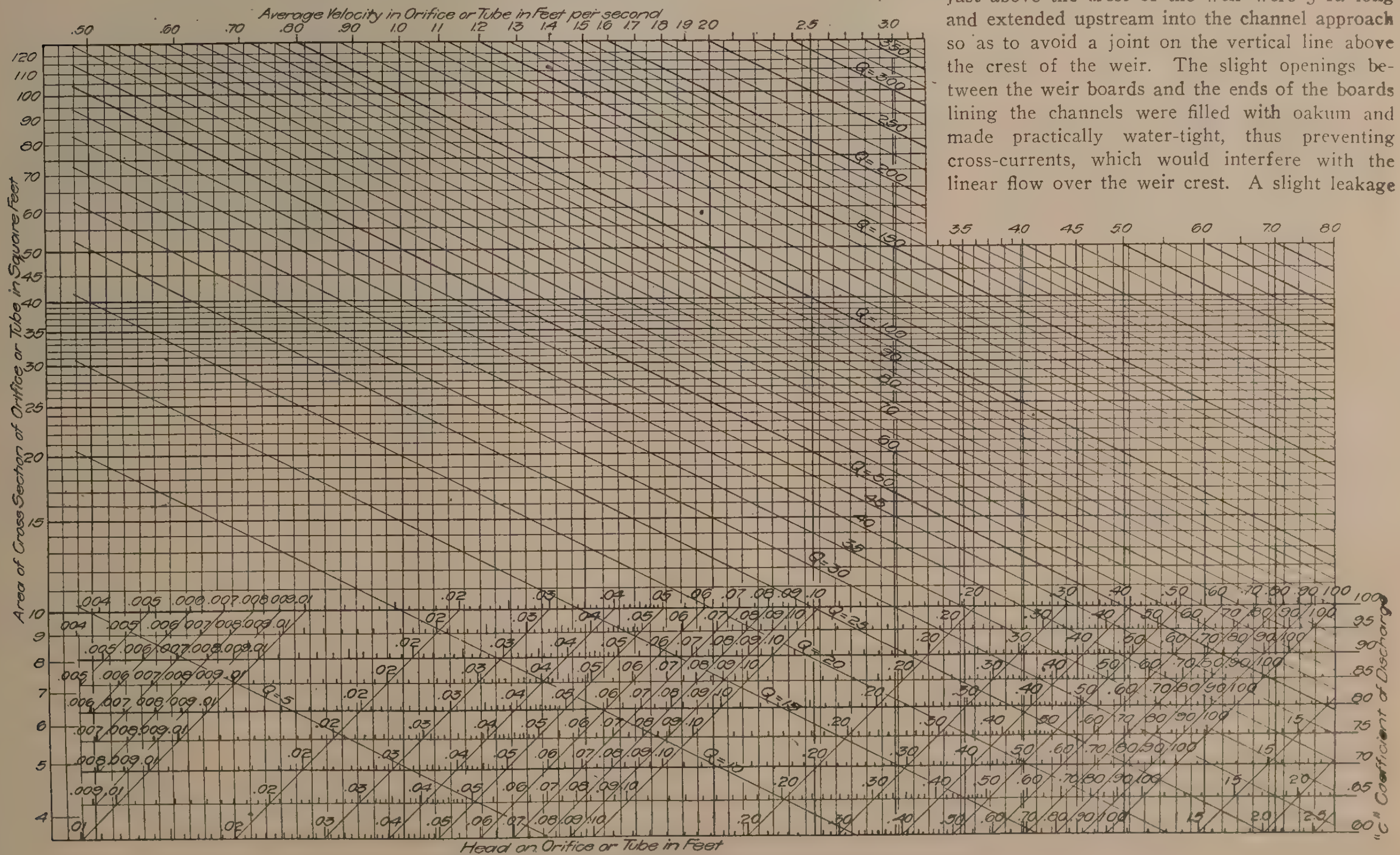


Diagram Giving Discharge Q for Submerged Tubes.

The bulkhead containing the tube experimented with was located about 24 ft. from the entrance to the flume.

The facilities for making a volumetric calibration of a weir whose length of crest is 10 ft. were not available, so the expedient of dividing the 10-ft. channel into three similar channels, each having a length of about 15 ft., and with a sharp-crested weir at the end of each channel, was used. In order to form a variable height of water surface on the downstream side of the bulkhead containing the submerged tube, the three sharp-crested weirs were made adjustable in height. The main feature of the weirs consisted of a fixed bulkhead, extending about 5 ft. above the bottom of the channel and a movable gate above the bulkhead, extending the full width of the 10 ft. channel and capable of being raised or lowered by means of two hand wheels, one on each side of the channel. The crest of the three weirs was made of a single plate of 6 x 3/16-in. wrought iron, bolted to the upper edge of the movable timber gate and beveled on the downstream side of the top edge to a thickness of about one-sixteenth of an inch. The joints around the weir openings were made water tight by means of gaskets formed of cloth insertion rubber tubing, one-half inch in diameter, the movable gate being drawn against the gaskets by means of lag screws passing through cleats adjustable vertically in the timber frame supporting the bulkhead and gate. Each of the three weirs had its own channel of approach about 15 ft. long. The sides of the channels were lined with one inch surfaced pine flooring boards so that all three of the channels of approach and weirs were practically alike, the only variation being that of slight difference in width of about one-half inch in the case of one weir. The board linings of the channel were extended downstream from the crest of the weir a distance of 1½ ft. and also below the crest of the weir, a distance of about 2 ft., so that the weirs had their end contractions suppressed and the filaments were guided in parallel planes after passing over the crest. The arrangement of the weirs with intervening spaces of about 8 in. between the ends allowed free entrance of air beneath the nappes.

The board linings around the crest of the adjustable weirs have to be removed each time the crests are adjusted. These boards were attached by counter-sunk flat head screws, and just above the crest of the weir were 5 ft. long and extended upstream into the channel approach so as to avoid a joint on the vertical line above the crest of the weir. The slight openings between the weir boards and the ends of the boards lining the channels were filled with oakum and made practically water-tight, thus preventing cross-currents, which would interfere with the linear flow over the weir crest. A slight leakage between channels of approach was unavoidable. The rubber tubular gaskets were horizontal at the top of the fixed bulkhead, and extended vertically upwards on each of the sides of the weir openings to the maximum height of water in the channel of approach.

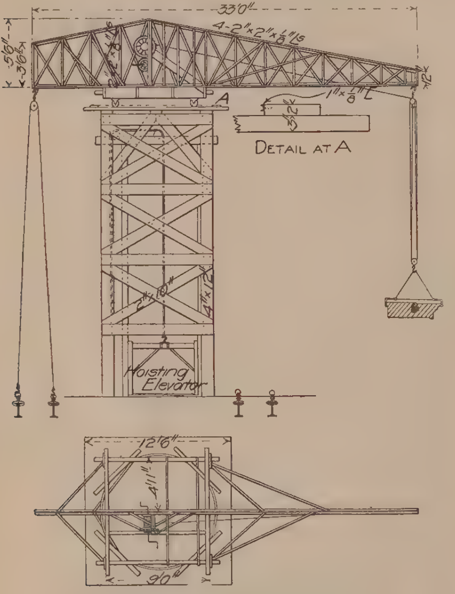
Before using the weirs for purposes of measurement they were tested for leakage by boarding above the crests of the weirs to a height of about

1½ ft. and filling with water. The tubular gaskets were very satisfactory and very little additional caulking was found necessary to make the weir channels water tight.

In the series of experiments with the submerged tubes 4 ft. square, the desired range of head on the tubes, zero to 0.4 ft., was possible without adjusting the height of the crest of the weirs.

The measurements of heads on the crest of the weirs were made by connecting the weir channels with still water basins located below the weirs, as shown in the illustrations, and taking readings of the elevations of water surfaces in the still-water basins by means of accurate hook gages. The connections between the weir channels and still-water basins were made by one-inch wrought iron pipes and one-inch rubber tubes. The tubes were attached to the basins by means of malleable iron unions and were easily disconnected. By lowering the outlet end of a tube and allowing free circulation of the water, the air in the connecting pipe and rubber tube could be easily removed.

The connections with the weir channels were made at points 8 ft. upstream from the crest of the weirs, 5 ft. above the floor of the channel and about 2½ ft. below the crest of the weirs.



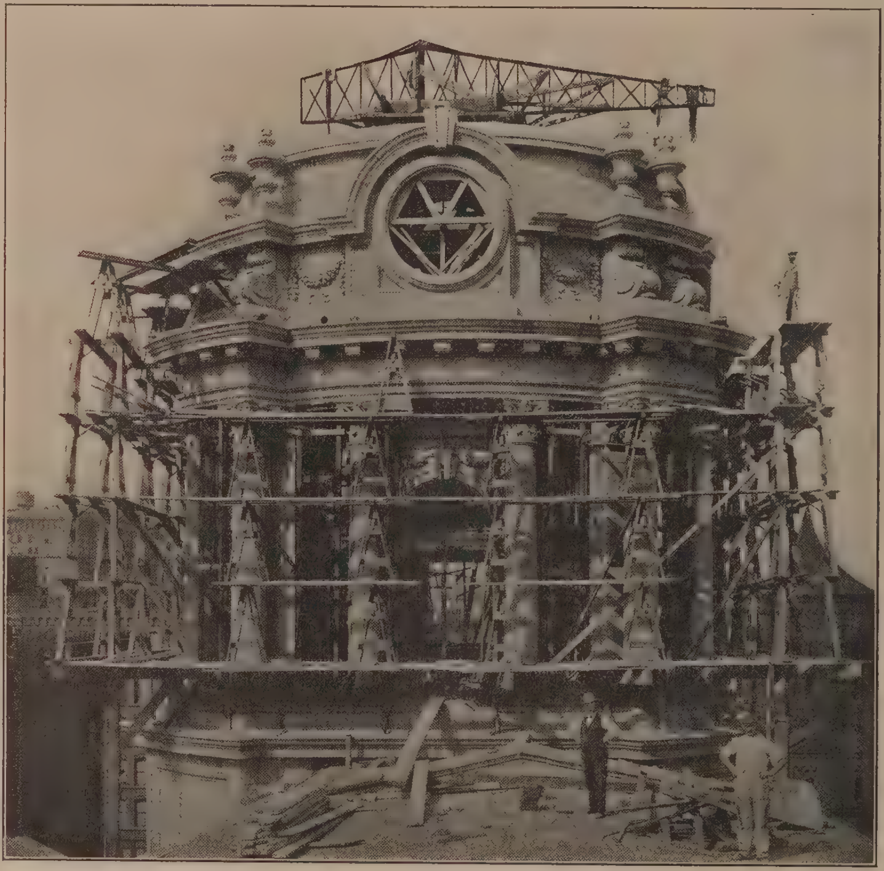
**Details of Turntable Crane.**

The plan followed was to study the simple forms of submerged tubes, 4 ft. square and of length varying from 0.31 to 14 ft., finding the losses of head and coefficients of discharge for the various rates of flow through the tubes. The entrance to the tubes of various lengths was then modified by introducing curved approaches, elliptical in form, and thus suppressing the contraction first on the bottom, second on the bottom and one side, third on the bottom and two sides, and fourth on the bottom, two sides and top. One of the illustrations shows the details of the submerged tube 2½ ft. long with the curved approaches on the various sides.

The submerged tubes with entrance square corners and of lengths varying from 0.31 to 14 ft. have been designated by series. The letters *a*, *b*, *c*, *c′* and *d* have been used with the various lengths or series numbers to designate the entrance condition. The subscript *c′* has been used to designate the same entrance conditions as the subscript *c*, but the outlet condition has been varied by the construction of a bulkhead to the sides of the channel at the outlet of the tube. One of the drawings show the forms of entrance and lengths of tubes used in the experiments.

The head, $h$, on the submerged tube has been taken as the difference of level between the surface of water above the bulkhead containing the submerged tube and the surface of the water at the outlet of the tube. The table gives a summary giving the coefficients of discharge for the various submerged tubes used in the experiments.

One of the diagrams is a graphical solution of the equations $Q = Ca\sqrt{2gh}$ and $Q = av$, used for submerged tubes and orifices. The ordinates on the left side of the diagram represent values of the cross-sectional areas $a$ and vary from 4 to 120 sq. ft. The abscissas marked on the lower part of the diagram represent values of the head $h$ on the submerged tube or orifice. These values of $h$ have been platted on horizontal lines, each line representing values corresponding to a certain value of the coefficient of discharge, $c$, as shown on the lower right hand part of the diagram. The abscissas marked on the upper horizontal line of the diagram represent values of the average velocities $v$ in the cross-sectional area $a$ of the orifice or tube. The oblique lines in the upper part of the diagram represent values of the discharge $Q$. The diagram is used like an ordinary diagram, having rectangular coordinates, except that the line on which the abscissas $h$ is to be read varies in position and depends on the value of $C$.



**Turntable Crane on Tower of New York Police Headquarters.**

Coefficient of Discharge for Flow Through Submerged Tube, 4 Ft. Square, for Various Lengths,

Losses of Head and Forms of Entrance and Outlet.

| | Entrance | Length of Tube in Feet. | | | | | | |
|---|---|---|---|---|---|---|---|---|
| h, Ft. | and Outlet. | 0.31 | 0.62 | 1.25 | 2.50 | 5.00 | 10.0 | 14.0 |
| 0.05 | Square | .631 | .650 | .672 | .769 | .807 | .824 | .838 |
| | a | .672 | .... | .... | .742 | .810 | .... | .848 |
| | b | .740 | .... | .... | .769 | .832 | .... | .862 |
| | c | .834 | .... | .... | .769 | .875 | .... | .890 |
| | c′ | .... | .... | .... | .... | .... | .... | .875 |
| | d | .948 | .... | .... | .943 | .940 | .927 | .931 |
| 0.10 | Square | .611 | .631 | .647 | .718 | .763 | .780 | .705 |
| | a | .636 | .... | .... | .698 | .771 | .... | .801 |
| | b | .685 | .... | .... | .718 | .791 | .... | .813 |
| | c | .772 | .... | .... | .718 | .828 | .... | .841 |
| | c′ | .... | .... | .... | .... | .... | .... | .830 |
| | d | .932 | .... | .... | .911 | .899 | .892 | .893 |
| 0.15 | Square | .609 | .628 | .644 | .708 | .758 | .779 | .794 |
| | a | .630 | .... | .... | .689 | .767 | .... | .803 |
| | b | .677 | .... | .... | .708 | .787 | .... | .814 |
| | c | .765 | .... | .... | .708 | .828 | .... | .839 |
| | c′ | .... | .... | .... | .... | .... | .... | .829 |
| | d | .936 | .... | .... | .910 | .899 | .893 | .894 |
| 0.20 | Square | .609 | .630 | .647 | .711 | .768 | .794 | .809 |
| | a | .632 | .... | .... | .694 | .777 | .... | .819 |
| | b | .678 | .... | .... | .711 | .796 | .... | .833 |
| | c | .771 | .... | .... | .711 | .838 | .... | .856 |
| | c′ | .... | .... | .... | .... | .... | .... | .846 |
| | d | .948 | .... | .... | .923 | .911 | .906 | .905 |
| 0.25 | Square | .610 | .634 | .652 | .720 | .782 | .812 | .828 |
| | a | .634 | .... | .... | .705 | .790 | .... | .... |
| | b | .683 | .... | .... | .720 | .809 | .... | .... |
| | c | .779 | .... | .... | .720 | .854 | .... | .... |
| | c′ | .... | .... | .... | .... | .... | .... | .... |
| | d | .965 | .... | .... | .938 | .928 | .... | .... |
| 0.30 | Square | .614 | .639 | .660 | .731 | .796 | .832 | .850 |
| | a | .639 | .... | .... | .... | .... | .... | .... |
| | b | .689 | .... | .... | .... | .... | .... | .... |
| | c | .788 | .... | .... | .... | .... | .... | .... |
| | c′ | .... | .... | .... | .... | .... | .... | .... |
| | d | .984 | .... | .... | .... | .... | .... | .... |

The diagram may be used in a variety of ways: Values of $C$ and $h$ may be assumed, as, $C = .75$ and $h = .10$, the vertical line through $h = .10$ will intersect lines representing $Q$, any value of which may be selected, and the corresponding value of $a$ determined by the horizontal line through the point of intersection of the vertical line and the line representing $Q$, as, for $Q = 50$, $a = 26.3$. By producing the vertical lines to the horizontal line on the upper part of the diagram values of $v$ may be determined, thus, for the above example, $v = 1.91$. The diagram may also be used by assuming values of $a$ and $Q$ or $v$ and $Q$ and the resultant values of $h$ read on the horizontal lines corresponding to some value of $C$, as shown on the lower right-hand part of the diagram.

The diagram will be found of special service in preliminary study where it is desired to study the relative effects of changes in a portion of the equation $Q = Ca\sqrt{2gh}$, as the effects on $h$ of certain changes in $C$ for a given area of cross-section and discharge.

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A Railway Motor-Car Test for long-continued high speed was made by the Union Pacific on Aug. 22 between Omaha, Neb., and Denver, Colo. The total distance of 570 miles was covered in 16 hr. and 34 min. The schedule time of the regular fast train is 17 hr. 15 min.

## A Special Turntable Crane.

The new police headquarters building now under construction at the northwest corner of Grand and Center Sts., New York City, is a massive stone and brick structure surmounted by a clock tower which rises about 40 ft. above the roof of the fifth story, as shown in an accompanying illustration. This tower is two stories high and will be capped by a hemispherical dome supported on steel trusses about 200 ft. above the building foundations. The material hoist for the building rises in a shaft under the dome. The construction of the tower up to the level of the architrave presented no unusual difficulties; above this level, however, the necessity of continuing the material hoist up to the base of the dome to serve the masons on the brickwork inside the tower and the fact that a boom derrick could not be used, owing both to lack of room inside the tower and the absence of guying-points, required the construction of a special revolving crane which was designed, built and erected on the work in three days.

This crane, which is clearly shown in accompanying illustrations, consists of a horizontal truss 33 ft. long, of the form shown in the drawings, mounted near its one-third point on a revolving table which is supported in turn at the level of the dome base on a 3-in. plank platform on top of the hoisting elevator framework, the latter having been specially braced and reinforced to carry the additional load. The truss is made up of 2x2x⅛-in. angle bars, four being combined in cruciform section for the upper and lower chords, two being used for all verticals, and single bars forming the X-bracing. The turntable consists of a frame, 9 ft. square, made of 3½x12-in. yellow pine beams placed on edge and braced across the corners with shorter pieces of the same sectional size. The four cast-iron wheels on which the table turns are 6 in. in diameter with 1½-in. flat face. They are pivoted to the lower edges of the corner pieces of the framework, the pivot point in each case being directly above the center of the wheel. A plank circle, about 9 ft. 10 in. in inside diameter and faced on its inner edge with ⅛x1-in. channel bar with the flanges turned toward the wheels, is nailed to the 3-in. platform to guide the turntable wheels which rolls directly on the planks.

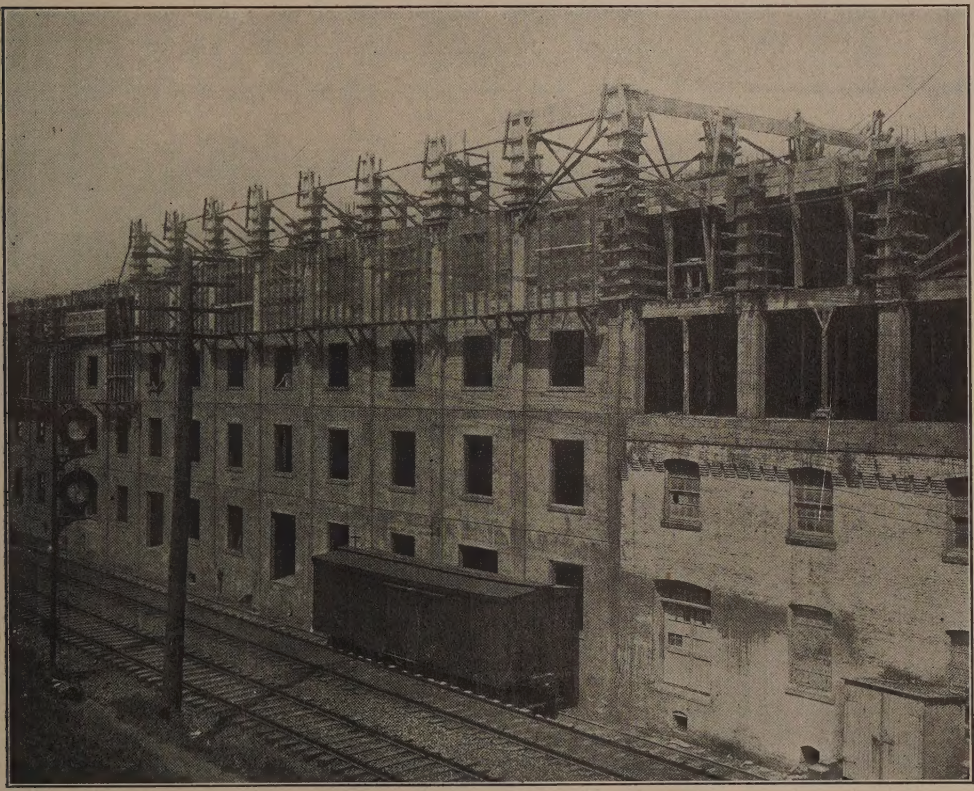
The manually-operated winch is fixed at one side of the truss over the turntable bearings on one side of the winch, being attached to a vertical truss member and those on the other side to a pair of vertical angles held in place at top and bottom by struts and braces in horizontal planes, as indicated on the drawings. To balance the overturning moment caused by the weight of the long arm of the crane and its load, the short arm was anchored to the beams of one of the tower floors by a tackle, as shown on the sketch.

All the stone and terra-cotta placed above the columns was delivered to the upper floor of the tower, about at the level of the architrave by the material hoist, which at the same time served the brick masons building the tower lining. The stone ornaments, some of which weighed over 1½ tons, were run on to the hoist at the first floor level on a four-wheel truck; when the hoist reached the level of the tower floor the truck was run off the hoist platform and across the floor to the inside of the wall opposite its final position outside; the stone was then hoisted, swung over the wall top and lowered to place with the crane tackle.

The crane truss was laid out full size on one of the lower floors of the building where it was riveted up in sections which could be easily handled. These sections were then carried up to the top of the hoist tower where the crane was assembled in place.

The building was designed by Messrs. Hoppin & Koen, architects, New York City, and Messrs. Gillespie & Walsh are the general contractors. Mr. Michael J. Garvey, superintendent, designed and erected the crane and other incidental apparatus described.

An Earth Current of unusual strength was recently experienced on the telegraph lines of the Canadian Pacific Ry. in Western Ontario and Manitoba. The current was strong enough to set fire to the switchboard at Fort William.



Plaster Mill Buildings, Showing Brickwork of Building 5 A.

## New Buildings of the King Plaster Mills, Staten Island.

Buildings 5 and 5A, now under construction as a part of extensive improvements to the plaster mills of J. B. King & Co., at New Brighton, on the north shore of Staten Island, New York, are five-story reinforced concrete structures occupying contiguous and irregular areas, as shown on accompanying plans, and designed to support floor loads of 200 to 600 lb. per square foot. They are located immediately on the bank of the Kill von Kull, a navigable stream, with their first floors about 5 ft. above mean high water. Behind the buildings is the right-of-way of the Staten Island Rapid Transit R. R., which provides a switch track for the several buildings of the plant. Behind the two buildings under consideration the tracks are about 4 ft. below the main floors.

Both buildings are founded on piles, 8 to 30 ft. penetration, driven through three sorts of material; riprap along the south side where a fill had been made by the railroad company; good soil outside the riprap; and old cribwork along the water's edge. In Building 5, where a floor load of 600 lb. per square foot is provided for on all floors, sixteen piles in general were driven for each interior column. In 5 A, which is designed for 200-lb. floor load per square foot on all floors except the sixth, where 600 lb. is provided for, there are in general six piles under each column and a continuous foundation girder distributing part of the column loads on intermediate piles.

*Building* 5.—The column footings in Building 5 are of the truncated pyramidal form and are in general 8 ft. square with their bases about 8 ft. below the first floor. The pile heads are embedded in the concrete 6 in., and 3 in. above their tops is a double series of ⅞-in. Ransome bars about 6 in. on centers each way. Owing to the unstable character of the foundation soil along the water front the three lines of interior columns immediately back of the north wall of the building were tied together and to the north wall columns at the tops of the footings by ties extending from the wall inward. Each tie is made up of two 1-in. Ransome bars encased in concrete to protect them from corrosion.

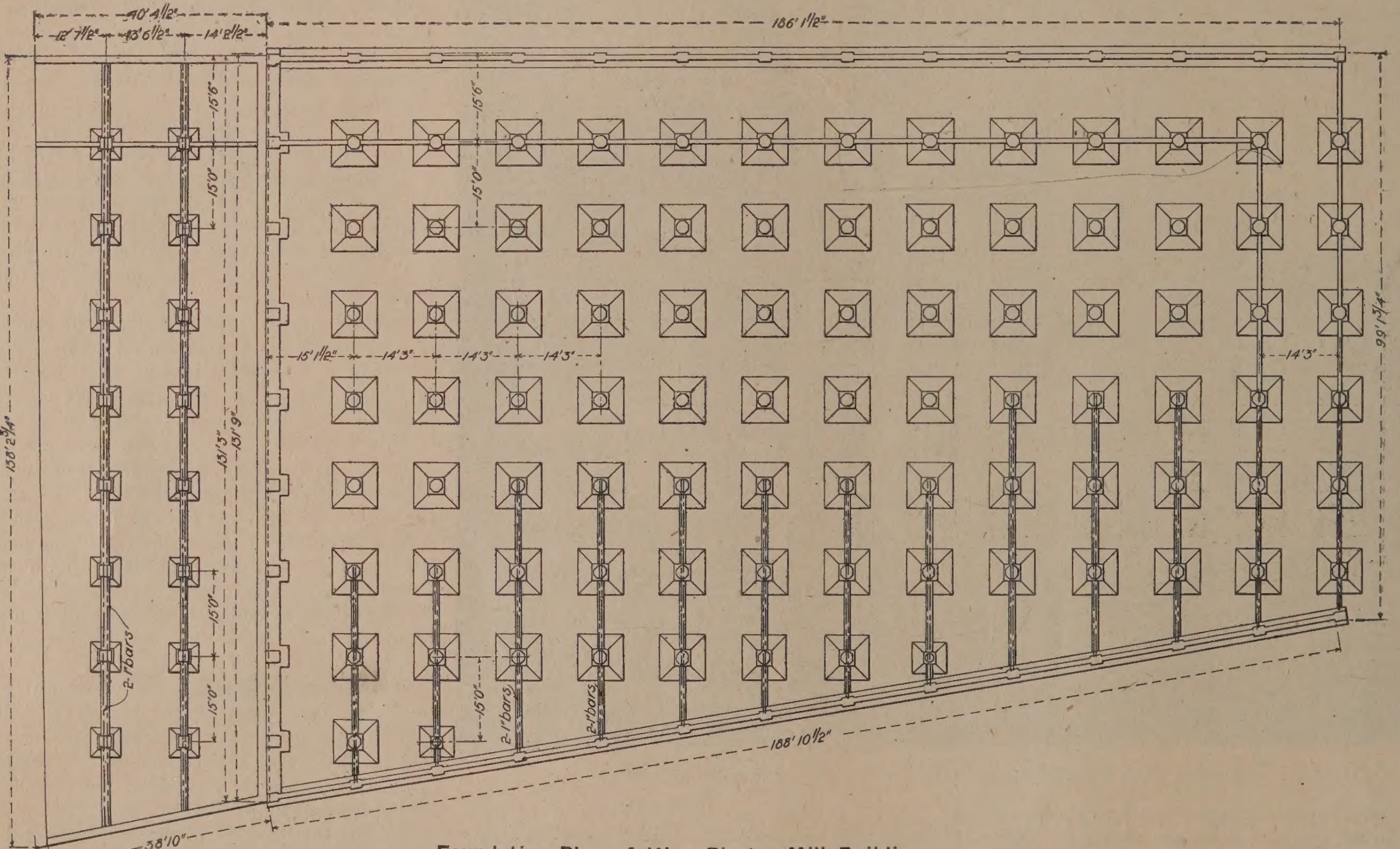
There are six complete lines of interior columns extending longitudinally through the building, the lines being 15 ft. on centers and the columns in each line 14¼ ft. on centers; besides these there are two incomplete lines similarly spaced, as shown on the floor plan. With a few exceptions the interior columns are octagonal in all except the fifth story, where they are square with the corners slightly chamfered. The typical interior column has the following dimensions and reinforcement in the various stories, the size being given as the side of the enclosing square: Below the main floor, 34-in., four 1-in. square rods and a spiral of ¼-in. wire 31 in. in diameter with 2-in. pitch; first story, 30-in., four 1¼-in. round rods, 7/16-in. wire, 27-in. spiral, 3-in. pitch; second story, 26-in., four 1¼-in. rods, ⅜-in. wire, 23-in. spiral, 2¼-in. pitch; third story, 22-in., four 1-in. round rods, ⅜-in. wire, 19-in. spiral, 3-in. pitch; fourth story, 17-in., four 1-in. round rods, 5/16-in. wire, 14-in. spiral, 2¼-in. pitch; fifth story, 12-in., four ½-in. Ransome bars, ¼-in. hoops 12 in. on centers. The vertical rods in each column are continuous through the first and second stories; at the third floor there are 1-ft. gas pipe sleeves and the rods above are continuous through the third and fourth stories. In addition to the vertical reinforcement mentioned there there are also in each column in each story where spirals are used, three or four small rods to which the spiral is wired to preserve its pitch.

These were not considered in designing the columns.

The north and south wall columns are rectangular in form, varying from 20 x 24-in. below the first floor and in the first story, to 18 x 24-in. in the second and third stories, and to 16 x 24-in., in the fourth and fifth stories. The reinforcement is four vertical Ransome bars, ⅞ to ½-in., with ¼-in. hoops 12 in. on centers. Across the west end of the building the usual wall columns are replaced by interior columns to provide for future extension in that direction. The east wall columns are carried up to the third floor beside a brick wall which will be referred to later. They are nearly square and somewhat heavier than the north and south wall columns. The first and second story columns are 12 ft. 6½ in. long from floor to floor. In the third and fourth stories they are 2 ft. shorter and in the fifth story they vary owing to the pitch of the roof.

The main floor girders extend longitudinally through the building and are in general 10 in. wide and 20 in. deep not including the 4¼-in. floor slab. The reinforcement consists of two 1⅛-in. and three 1-in. Ransome bars with ⅜-in. stirrups properly distributed. The floor beams are 4 ft. 9 in. on centers and divide the floor slab into panels, 4 ft. 2 in. x 14 ft. 2 in. They have three 1-in. twisted rods and 5/16-in. stirrups. The floor slab is 4¼ in. thick, including a ¾-in. finishing coat, and is reinforced across the beams with 5/16-in. Ransome bars, 30 ft. long and 5½ in. on centers. Each floor panel is bonded longitudinally with two ⅜-in. bars.

The typical wall girders, which are important only during construction when they serve as ties and carry the weight of the wall forms and concrete, are 4 in. wide and 20 in. deep and have one 1-in. bar and ⅜-in. stirrups. When these girders are poured six 5/16-in. rods, 2 ft. long are inserted in the concrete and left projecting from the upper surface of the beam to bond the wall to it.



Foundation Plan of King Plaster Mill Buildings.

The building is divided about on its north and south center line on all floors by a 6-in. concrete fire wall set into vertical 4 x 1-in. recesses in the columns. This wall is reinforced by vertical ⅜-in. bars 24 in. on centers and horizontal ⅜-in. bars 18 in. on centers. Wherever partition walls are required, and many are necessary to form bins, 1 x 4-in. recesses are left in the proper columns and vertical bond bars are inserted in the girders and left projecting above the floor to hold the bottom of the wall. Slot openings are left in the floor above all partition walls through which to pour the wall concrete.

*Building 5 A.*—This building is immediately east of and adjacent to Building 5, and occupies a site upon which formerly stood a two-story brick building of mill construction, in fact, the north, south and west walls of the old two-story building form the walls of the lower two stories of the new building. Its east wall is the west wall of a five-story brick mill building. This wall is 20 in. thick and is securely founded on piles.

Structurally, Building 5 A is in general much like Building 5. It is much lighter, however, its floors having been designed to carry 200 lb. per square foot, except the second, which is designed to support 600 lb. per square foot. It is about 40½ ft. wide and 135 ft. long, both these dimensions varying owing to the irregularity of the site. There are two longitudinal lines of interior columns 15 ft. on centers in general in each line, dividing the building into three longitudinal bays, 12 ft. 7½ in., 13 ft. 6½ in., and 14 ft. 2 in wide. The transverse girders carry floor beams 4 ft. 4¾ in. to 6 ft. 4 in. on centers. The floor slabs are 4 in. thick except in the east bay of the second floor which is 5½ in. There are no wall columns on the east side, the ends of all girders on that side being set into the brick party wall 9 in. A recess 4 in. deep is also cut into the brick wall to receive the floor slabs. On the west side the same construction is used up to the top of the brick wall; above this there are wall columns resting on the brick wall and bonded to the wall columns of Building 5.

The north wall of the old two-story brick building had broken loose from the side walls, owing to the unstable character of the foundation material along the water front, and to secure this and guard against slipping, the interior column foundations were securely tied together throughout the whole length of the building by longitudinal lines of 1-in. bars encased in concrete and resting on top of the foundation girders. The base of the north wall was firmly secured to this system at numerous points by tie rods and plates.

*Construction Details.*—Building 5 A was started first but work on the two buildings is now being carried forward simultaneously. The concrete used in the columns is a 1:2:3½ mixture of Alpha or Atlas Portland cement, Cow Bay sand, and beach gravel. For the girders, beams, floors and walls a 1:3:5 mixture was used. The sand and gravel is delivered on the work by the owners from their large sand washing and drying plant at Port Washington, Hempstead Harbor, on Long Island Sound. It is brought to the bulkhead line in front of Building 5 in lighters; shoveled into buckets; and hoisted to a platform at the level of the second floor where it is dumped into large two-wheeled hand carts and hauled to storage piles on the second floor. These storage piles have at times been so deep that the load on four bays amounted to 1,200 lb. per square foot, but there was no noticeable deflection. The derrick hoist is operated by a Lambert portable single-drum hoist driven by a 7½-h.-p., 200-volt, G. & C. Electric Co. motor. The sand being thoroughly dry is closely compacted, 86 per cent. of it being equal to 100 per cent. of the same material undried. The cement is delivered from cars on the switch track to the first floor of Building 5, where it is piled near the mixer.

The mixer, a Ransome machine, is placed near the center of the first floor. Cement is delivered by wheelbarrow on an elevated platform beside the mixer and sand and gravel are introduced through a hopper on the second floor, being brought from the storage piles in wheelbarrows. The mixer delivers to a Ransome self-dumping bucket elevator with a hopper on every floor. Both mixer and elevator hoist are operated by a 40-h.-p., 200-volt, motor made by the Eddy Electric Manufacturing Co., Windsor, Conn. The motor, Mundy friction hoist, and mixer are placed in a line in the order mentioned. The motor is belt-connected to the shaft of the hoist and the mixer is chain-driven by a sprocket on the hoist shaft, the drum being thrown in or out by a clutch. One man operates the hoist and attends the motor and mixer, a system of electric signal bells having been arranged for use with the elevator. Current for the motors is supplied from the private plant of J. B. King & Co.

The wire for the column helices is formed in coils of the proper diameter at the shops of the contractor. On the work it is formed into helices with the proper pitch on a long framework, T-shaped in section, the T being the right size to fit the inside of the spiral and one of the three leaves being hinged to permit the easy withdrawal of the completed helix. The pitch is adjusted according to marks on the frame and longitudinal rods are wired inside to preserve the spacing.

The forms present no unusual features. Cement block spreaders with a bolt hole through the center are used in the interior walls only. The small pieces needed in the forms are cut in a shop on the first floor in which are installed a circular saw, boring machine, and planer, operated by a 10-h.-p. 250-volt, General Electric motor. Several blind windows are to be filled with 4-in. concrete slabs made on the floor in front of the openings in tin-lined forms. A piece of rod bent in U-form and securely fixed in the back of each slab serves as an attachment for hoisting them into place with a hand tackle and portable derrick frame.

The buildings described were designed and are being built by the Turner Construction Co., New York City.

## Book Notes.

It is rather difficult to understand just what purpose Messrs. W. S. Franklin and Barry MacNutt had in mind in writing their "Elements of Mechanics," for its text is such a contrast to that of successful textbooks on the same subject as to be startling. The orderly development of the principles of pure mechanics which is necessary to enable the student to grasp the fundamental facts of the subject is apparently abandoned in a measure in favor of somewhat disconnected sections on problems in applied mechanics. Prof. Franklin is the author of other books of high merit, which make it surprising to see his name attached to one of such a disjointed character. It may be that it will prove a useful class-room adjunct to a more vigorous course given by other means, but from the viewpoint of the usual class-room methods it seems about as useful as a text-book as the works on light and heat written by the late P. G. Tait and still regarded as unequalled examples from a master's hand of how an elementary book may become a mystery. And yet there are a good many sections of Messrs. Franklin and MacNutt's book that are inspiring to an ambitious beginner in physics, even if they have a tendency toward superficiality. (New York, Macmillan Co., $1.50.)

A book on detailing steelwork has been written under the title of "Notes on Construction in Mild Steel," by Mr. Henry Fidler, M. Inst. C. E. The preface is a particularly good statement of the complexity of the subject, which is the most troublesome part of structural steel design. The author repeats the familiar remark that the ability to analyze correctly the stresses in a roof truss and the ability to design a good riveted connection are not quite one and the same thing. Close study, observation and experience are needed for a draftsman to become a competent detailer and even then the knowledge of the real expert has not been acquired. This must be gained by becoming familiar with the treatment the steel receives in the furnaces, mills and shops, in order that the real degree of approximation of practical methods to theoretical assumptions may be known, and by learning in the field the procedures of the erection gangs and the troubles they have to overcome. The American reader should recognize, also, that the young engineers of Great Britain have not taken courses in technical colleges to the almost universal extent customary in the United States, and a British book on steel detailing is consequently required to go into matters that seem rather out of place to those who, as undergraduates, received a thorough grounding in metallurgy and the properties of materials. For this reason the numerous tables of tests of steel in the first half of the book have a value to the British draftsman which an American engaged in similar work will find some difficulty in appreciating. Nearly a fifth of the book is devoted to the chapter on the manufacture, physical and chemical qualities of mild steel and the second chapter gives in condensed form the same class of information regarding rolled sections that American engineers are accustomed to obtain from mill handbooks. The remainder of the volume is a discussion of detailing girders, columns, roofs and marine steelwork and of the methods of protecting steel surfaces. The numerous illustrations are mainly from drawings prepared for regular engineering purposes, and throughout the book are useful notes of special devices or methods of a kind that rarely get into print. The book is one that should prove valuable in any large detailing office and in the libraries of technical colleges. (New York, Longmans, Green & Co., $5.00.)

Mr. Allen Hazen has rendered a distinct service to all who wish to know about municipal water supplies by writing "Clean Water and How to Get It," a book which is both authoritative and free from the technicalities that interest sanitary and engineering specialists only. At the present time considerable harm is occasionally done by city authorities who, with the best of intentions, are misled by half-knowledge into undertaking works that should never be constructed. A careful reading of this book should prevent such ill-judged decisions; it has been written particularly for mayors and aldermen who have been chosen to office from walks of life in which they have had no water-works experience, and can be aided in serving their cities by clear statements of principles, illustrated by examples from large and small works which are fulfilling their purpose well. The book opens with a description of impounding reservoirs and explains the changes that take place in the quality of water stored in them under various conditions. A brief chapter on supplies from small lakes points out their similarity to those obtained by impounding streams. The chapter on supplies from the Great Lakes summarizes the views of the leading authorities concerning such a source, and calls attention to their danger on account of sewage contamination. The author states plainly, and deserves hearty applause for doing so, that while Chicago and Cleveland have succeeded in getting better water by spending millions in driving tunnels far out under the bottom of the lakes to secure supplies free from contamination, they have failed to get thoroughly good water. In the chapter of supplies from large rivers the author makes statements regarding sewage purification that are particularly important in these days of a growing public demand for the universal purification of sewage discharged into rivers. "The fact must be fully recognized," he asserts, "that the discharge of crude sewage from the great majority of cities is not locally objectionable in any way to justify the cost of sewage purification. Looking at the matter as one great engineering problem, it is clearly and unmistakably better to purify the water supplies taken from the rivers than to purify the sewage before it is discharged into them. The volume to be handled is less and the cost per million gallons of purifying water is much less than the cost of purifying sewage. It is also very much more effective to treat the water, because the methods of water purification are more efficient in stopping germs of disease than are the methods of sewage purification." Ground-water supplies are discussed in one of the most interesting chapters in the book, which explains the effect of different conditions on their quality and quantity, and the results of the action of water on iron pipes is described in an equally important chapter containing considerable information not readily found elsewhere. The next five chapters are on water purification and related subjects and are excellent outlines of what are now considered the best ways of purifying different waters and of removing turbidity, color and iron. That these methods can be improved as conditions demand is stated by Mr. Hazen in the following confident forecast: "It may be reasonably anticipated that far more efficient methods of purification will come to be used in course of time and in discussing the advantages and disadvantages of polluted waters after purification for use by cities through a long term of years with steadily increasing amounts of pollution, it will not do to consider only the methods of purification now in use. Better methods will be available for the more difficult service when they are needed." The subject of water pressures is given a chapter by itself and so is the designs and construction of works, and some wholesome advice is given in them which, if followed, would result in a gratifying reduction of charges or in improvement in service in some cities. The cost of waterworks is taken up briefly in another chapter. The chapter on the use and metering of water and that on the financial management of public works are different in character from the rest of the book, as they are mainly arguments in favor of the author's opinions concerning subjects which offer a wide field for controversy. (New York, John Wiley & Sons, $1.50.)

## Letters to the Editor.

### LATTICE BARS IN THE QUEBEC BRIDGE.

SIR: I notice in The Engineereing Record and elsewhere that a discussion has started on the question of secondary stresses, and yet I doubt if any of us for a moment believe that secondary stresses had anything to do with the failure. One particular lower chord section seems to be considerably under discussion, while it is evident that other lower chords were of quite the same section, that is, some 760 sq. in., something like 16 in. radius vertically and approximately 19 in. horizontally, with stresses within limits that no one would expect to cause failure. But we are all confronted with the fact that these members had only 2.8 per cent. of the weight of the section of the member in lattice and its rivets, while it is a fact that our practice shows that of moderate sizes in lightest section, 8 to 15-in. channels, the weight of lattice and its rivets of a two-channel members varies somewhat, but is about 30 per cent. of the weight of the two channels; that is, practice uses about eleven times as much relative weight of lattice on small-size columns as were used on the compression members of the Quebec Bridge. That this may be the real cause of failure, I am not prepared to assert, but

I am prepared to hazard the opinion that none of us will build latticed members in excessive sizes with as small lattice bars and as few rivets in their connections in the future as we may have done in the past.

The relation of lattice to the member is not specified anywhere in any general specification. That the individual lattice with its rivets should have some relation to the stress of the member surely appeals to us as reasonable since the failure of the Quebec bridge, and yet no one has ever put in print anything that has come to my notice on the subject.

Horace E. Horton.

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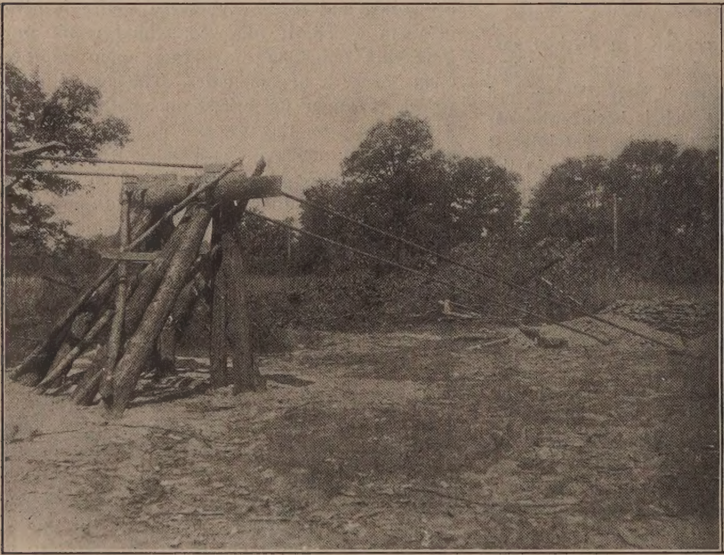
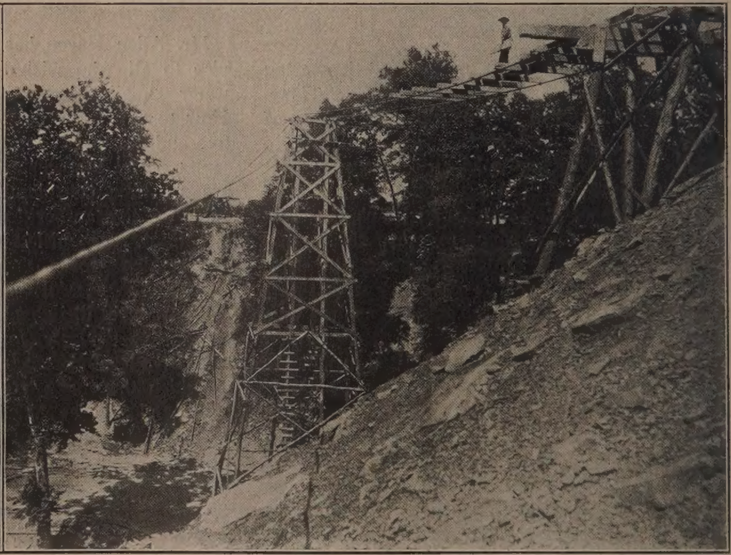
### Cost of Aerial Cableway in Making a Fill.

Sir: In making a single-track, 175,000-yd. fill, the Williams Bros. & Morse Co., of Cleveland, Ohio, saved $1,700 by using an aerial cableway in preference to a timber trestle. The fill was 400 ft. long and 95 ft. deep, between Stations 377 and 381 on the Lake Erie & Pittsburg Ry., now under construction from Cleveland to Pittsburg. A detailed, accurate account of all expenditures made on the cableway for material and labor was kept, and a careful estimate was made by Mr. A. E. Williams, of the Williams Bros. & Morse Co., of the least probable cost of a trestle which would have been necessary to meet the same conditions as were presented to the cableway.

Frequent accidents on cableways have rather shaken the confidence of contractors in this means of providing a bridge for the dump cars, but it seems that these mishaps have been due to improper construction and the use of cables that were not large enough to stand the heavy strains. But in this case the cableway has well withstood the loads that have been put upon it, as the anchors and wooden supports were made extra heavy, and a 2¼-in. Roebling galvanized bridge cable was used.



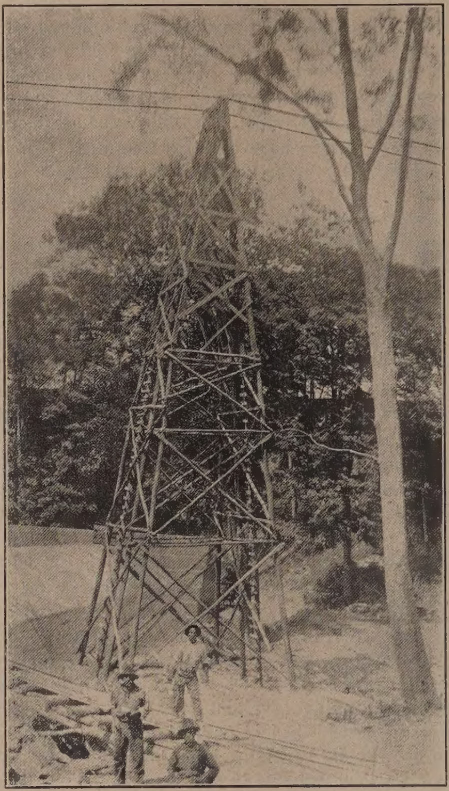
Method of Anchoring Cables.



General View of Cable Road.

The anchors on either side were 400 ft. apart, and an A-frame was erected to support the cable in the middle; this made the span 200 ft. The anchor at the north bank consists of a log 18 ft. long, 24 in. thick, imbedded in solid rock. Two eyebolts screw into the log and are fastened by heavy nuts over 8-in. cast-iron washers. Connecting with these eyebolts are two 10-ft. chains with 10-in. links made from 2½-in. iron. These chains were put in to keep the cables from twisting by covering the chains with heavy weights. Two 3-in. turnbuckles, with a spread of 3 ft., made the connections between the chains and the cables. The cables were leaded into the turnbuckles. These turnbuckles, which were forged for this work, were used for taking up the slack.

The anchor at the south end consisted of a log 25 ft. long and 24 in. thick, placed in a new fill of sandstone 10 ft. deep. Three-inch planks were driven in front of the anchor log, two eyebolts 22 in. long and 2½ in. in diameter were screwed into the anchor log and fastened with nuts over cast washers 8 in. in diameter and 2 in. thick. The eyebolts connected with clevises by means of 3-in. pins. The cables were leaded into these clevises. A rise of 1 ft. in 3 ft. brings the cables to grade and to the end timber supports.



The Central Tower.

The fill is being made from the south end of the cableway and bents are put in as the work advances to help support the weight of the cars. The support for the cables on the north bank is made of 10-in. logs resting on mud-sills. There are two 6-in. hickory rollers on which the cables rest to allow them to accommodate themselves to different lengths.

The A-frame which supports the cables in the center is made of two bents of four timbers each. The lower 50 ft. of the frame is made of 10-in. round timbers and the upper 42 ft. of 8x8-in. square timbers. The cables on the top of the A-frame are 8 ft. above grade. The bents rest on 10-in. mud sills. They have a batter of 1½ in. to a foot. The frame is 32x26 ft. at the bottom.

A train consists of from six to twelve 4-yd. cars. These are emptied at the south end of the ravine and are pushed out onto the cableway as fast as they are emptied. The car rails are spiked to ties which rest on stringers. These stringers rest on 8-ft. logs fastened to the cables with U-bolts. The cables are 7 ft. apart. Most of the fill is being made from a sandstone cut about a half mile distant. This sandstone has a slope a little steeper than 1½ to 1.

The following is the actual cost of the aerial cableway: 1,000 ft. of 2¼-in. Roebling galvanized bridge cable, $600; 2½-in. eyebolts and clevises for both ends, $108.30; 2 3-in. turnbuckles at north end, $120; 2 2½-in. iron chains at north end 10 in. long, $62.40; 4 8-in. cast washers 2 in. thick, $2.46; timber for A-frame, all bracing and cross ties (other timber obtained on ground), 3,200 ft. at $34, $108.80; lower 50 ft. of A-frame, round timber, 56 ft. long, bought in tree, $32; cost of team work for hauling round timber and pulling timber to place for erecting, $65; carpenter labor on A-frame and end bents on bank, $231.40; time of superintendent, getting material and overseeing work in general, $60; common labor in digging trenches for anchors and putting up cableway, $112; nails and iron in A-frame and bents, $29.40; total cost of cableway, $1,531.76.

A conservative estimate made by Mr. Williams of the probable cost of a timber trestle for this opening, figuring on square 8x8-in. timber cut from native timber: 98,000 ft. (including all uprights, planks for bracing, stringers, etc.), at $26, $2,548; labor for putting up trestle, $6 per M., $588; spikes, $98; iron drift bolts, $40; total cost of trestle, $3,274.

Saving effected by using cableway: Estimated minimum cost of trestle, $3,274; actual cost of cableway, $1,531.76; difference in favor of cableway, $1,742.24. Yours truly,

Cleveland, Ohio. J. D. Mooney.

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Tar-Macadam Roads are now being laid in several English localities by a method developed by the engineer of the Eton and Slough Rural District Council. After the upper layer of old macadam has been removed a coat of tar cement is spread and broken stone rolled into it until the tar shows on the surface. Such work has been done at about 42 cents per square yard.